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ENERGY ON GUAM

The Feasibility Of Utilizing Wastewater Discharge For Hydroelectric Power Generation On Guam

Guam. Coastal Zone Management Program

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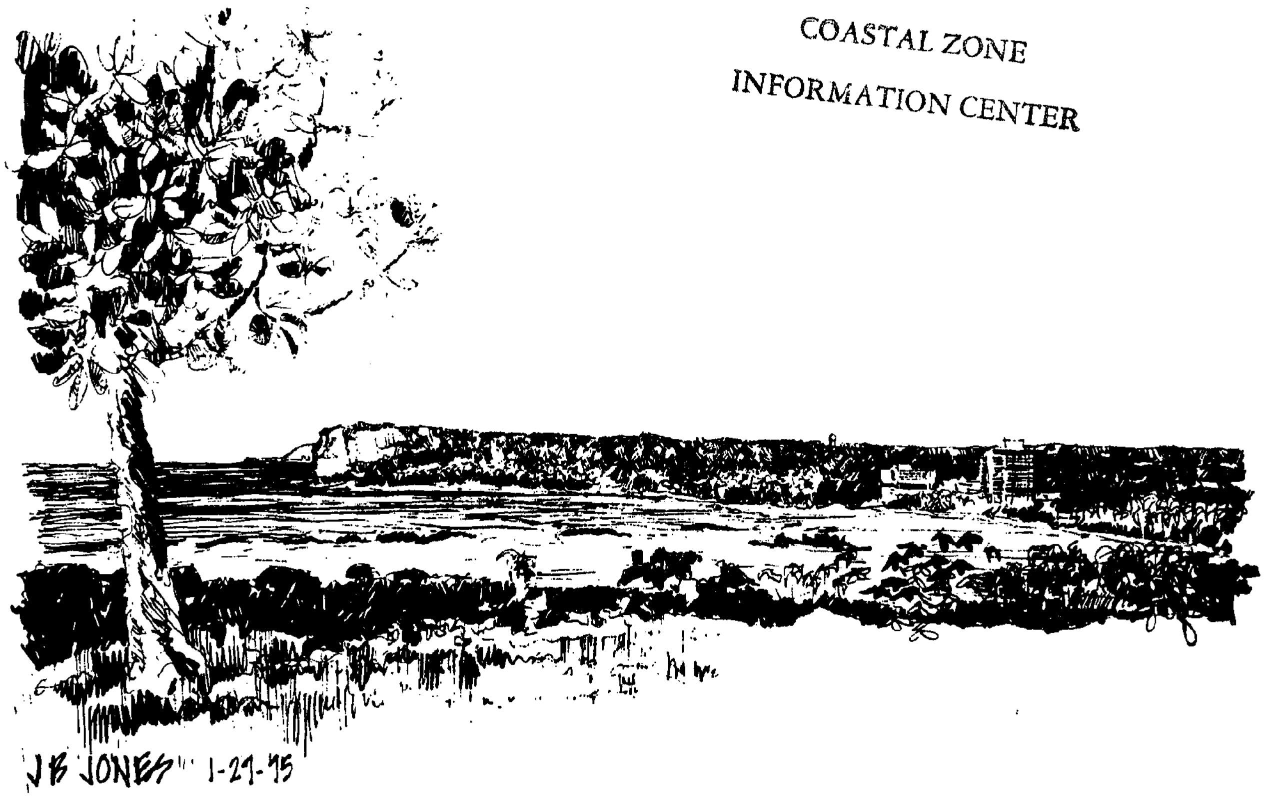

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GOVERNMENT OF GUAM

GEO TECHNICAL REPORT Nº 8103

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THE FEASIBILITY OF UTILIZING WASTEWATER DISCHARGE FOR HYDRO-ELECTRIC POWER GENERATION ON GUAM

A STUDY

CONDUCTED FOR

THE GUAM ENERGY OFFICE

GOVERNMENT OF GUAM

COASTAL ENERGY IMPACT PROGRAM

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by

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EXECUTIVE SUMMARY

Pacific Energy Management Consultants was contracted to study the feasiblity of the use of wastewater outfall for hydroelectric power generation on Guam. This study was funded under a grant provided by the Coastal Energy Impact Program and administered by the Guam Energy Office, Jay L. Lather, Director, under contract number C-0-3400015.

All island wastewater outfall sites were examined. With the exception of the Northern District Sewage System (NDSS), all sites were found to have inadequate head-volumetric flow characteristics. The NDSS system is capable of producing 50 kilowatts of electrical power. The NDSS already has in place many features of a hydroelectric power generating system, thus minimizing the initial cost of an installation. Although NDSS flow parameters fall below those required of traditional small hydro projects, centrifugal pumps can be used in a reverse mode, as turbines, to implement a reliable low cost generating system.

First cost of implementation is estimated at $109,250. The current annual value of electrical power produced is projected to be $43,800. The savings to investment ratio is 6.50.

The project can be funded through existing federal programs.

The project is technically feasible and yields significant economic and energy savings benefits. Project implementation is recommended.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF TABLES

Table

LIST OF ILLUSTRATIONS

Figure

SECTION I

INTRODUCTION

Hydroelectric power generation began in the nineteenth century and continues to the present day as a practical means of production of electric power. Because of its self-sustaining nature and freedom from dependence on conventional fossil fuels, it has received increased attention in recent years as an attractive alternate energy resource.

In hydroelectric power generation, water is passed through a turbine to produce rotary power which is in turn converted to electrical power in a generator. This train of energy conversion ending with the production of electricity begins with a water impoundment system which creates a volume and elevation of water of sufficient magnitude to achieve the desired result.

No matter how small, any mass which possesses elevation with respect to some reference level possesses energy; therefore any quantity of water trapped in an impoundment system can theoretically be used to generate electricity. Practical limitations in the form of the economics of design and construction of such facilities, compared to the power realized from their operation, tend to place lower limits upon the size of commercial or institutional generation systems. These limitations are usually expressed in terms of lower limits of volumetric flow and elevation or head between the impoundment site and generating system. It is important to realize that flow and head limits derive from economic considerations. It is possible to design turbine-generator systems for any non-zero flow or head; it is not, however, practical.

Several firms have developed stock turbine designs and thus lowered design and manufacturing costs; this has enabled the recovery of energy from otherwise economically marginal or unacceptable sites. Presently, such designs will operate in applications where flow is as little as 50 cubic ft. per second and head as small as 6.5 ft.[1] A generating facility operating with these minimum resources might conceivably provide sufficient electrical energy to support ten or fifteen middle class residences.

There is a trade-off on flow and head: small volumes at high head can be made to produce useful power; however present conventional hydroelectric turbine designs do not function below the levels mentioned. Guam's volumetric flows are far below the limit of conventional turbine design. It is unlikely that Guam will witness hydroelectric development in this conventional sense.

The question immediately forms: is there some other path to the production of hydroelectric power on Guam? The answer is yes. A wide variety of pumps are commercially available and many have been demonstrated to be capable of reliable reverse operation; i.e., being driven by a flow of water to produce shaft power rather than using shaft power to produce water flow. The variety available offers reasonable assurance that pumps can be found which match flows and heads found locally. Rugged pump designs are available at relatively low cost. In spite of the absence of suitable conventional devices, Guam still can make use of its water resources.

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[1]Smaller designs are commercially available; however, these devices are not considered appropriate or feasible in the context of this study. This point is more fully treated in Section III.

The second question forms: are there economically feasible sites on Guam? Other work in progress at the time of preparation of this study examines conventional sites: Guam's rivers, streams and existing or proposed water impoundment locations.[2] It is the purpose of this study to examine a less conventional source of hydro energy: the island's wastewater system.

Most of the island's wastewater system cannot be used for such application; there is not sufficient volume and heads are minimal (Appendix A). One notable exception is the Northern District Sewage System. Here approximately 2 million gallons of treated wastewater are daily discharged into the open ocean under a head of more than 250 feet. A rudimentary engineering calculation shows that this flow can produce small but useful amounts of electrical energy. The nature of the system offers hope that this production may be cost effective because the existing system possesses many of the features normally required in hydroelectric power generation; thus, the cost of the installation is reduced to that of the generating facility alone. The impoundment system, penstock, and other components are already in place.

Another inviting feature of the Northern District system is that the potential generating system site is located within approximately one mile of two possible users of electricity produced: the treatment plant itself and a conventional fossil fuel electrical generating plant (Tanguisson) operated by the Guam Power Authority (GPA). The electrical energy produced can be returned to the sewage plant or can be fed into the GPA distribution grid.

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[2]See: "Economic and Environmental Impacts of Low Head Hydroelectric Power Systems, in Guam," Guam Energy Office, 1981.

These possibilities have prompted this present study. Its purpose is to examine this method of hydroelectric power generation: to define the equipment and system required, to investigate operating characteristics, to quantify benefits and to thus establish feasibility.

SECTION II

GUAM'S WASTEWATER DISPOSAL SYSTEM

2.0 GENERAL:

Guam's disposal system collects wastewater from various areas of the island and, following treatment, discharges it into the Philippine Sea (see map). In most cases collection and treatment occurs at elevations near sea level and so rules out, by the lack of sufficient pressure, the use of this flow in hydroelectric power production (See Appendix A). The significant exception to this situation is the Northern District Sewage System. There, wastewater from the northern part of the island is collected and treated at a plant at Harmon Cliffline near Tanguisson Point; the fluid component is discharged into the Philippine Sea from an elevation of approximately 270 ft.

Although designed for a capacity of 12 mgd, plant statistics show a current processed daily volume of only 2.2 mgd. Cutover plans in which additional segments of the island's system will be channeled to the Northern Sewage Treatment Plant will, if implemented, increase the processed volume to some 6 mgd or more, but these plans are at present indefinite and at least three years (and perhaps much farther) in the future. Increasing daily volumes can be expected as the result of new connections to the line.

The volume of this system is far too small to permit it to be considered in terms of conventional hydroelectric power generation systems. However, as emphasized in the introductory remarks, any mass possesses potential energy by virtue of its elevation above some datum level. The volumetric flow and

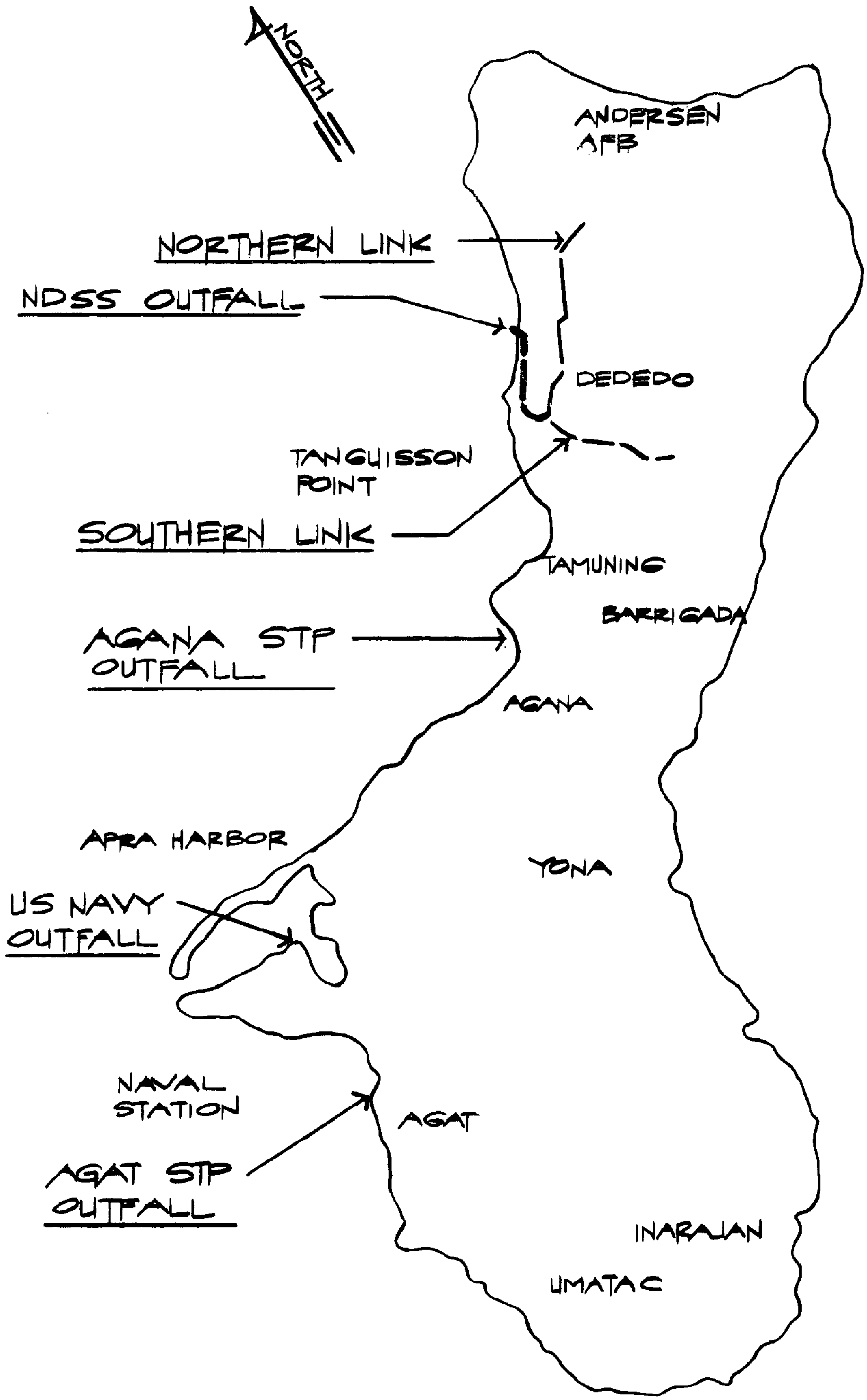

FIGURE 1
GUAM'S WASTEWATER OUTFALLS

head of the water mass of the Northern District outfall therefore represents a total potential power of 69 hp. The use of small pumps as turbines driving generators will permit the recovery of this energy, energy which is presently wasted. This ammount of power seems insignificantly small until expressed in economic terms: this power could produce $43,800 worth of electricity annually at present rates ($0.10/KWH).

2.1 THE SYSTEM:

Discharge from the Northern District Sewage Plant is carried by gravity through a single 48 inch (diameter) pipe for a distance of 948 ft. to Guam's northwestern cliff line. This segment of the line is buried to an average depth of approximately 30 feet. At the cliff line, at an invert elevation of 268.5 ft., the line intersects a manhole which provides access from the surface to the line. The remaining run of pipe from manhole to defuser section is a single 30 inch (diameter) buried pipe (See Figure 2).

The precipitous descent from cliff line to beach level carries the discharge from invert elevation of 268.5 feet to 13.3 feet in a pipe line distance of 3,220 ft. At the end of this segment, the pipe is buried some 6 feet beneath the surface. The final segment, from end of cliff descent to defuser section where discharge into the ocean occurs, is 3,170 ft.

The pipe itself is constructed of Reinforced Plastic Mortar (RPM), Techite T-2, manufactured by the Amoco Reinforced Plastics Company. The segment of line carrying wastewater from cliff line (invert elevation 268.5 ft) to bottom of cliff (invert elevation 13.3 ft.) is Techite T-2 150 lb/in^2

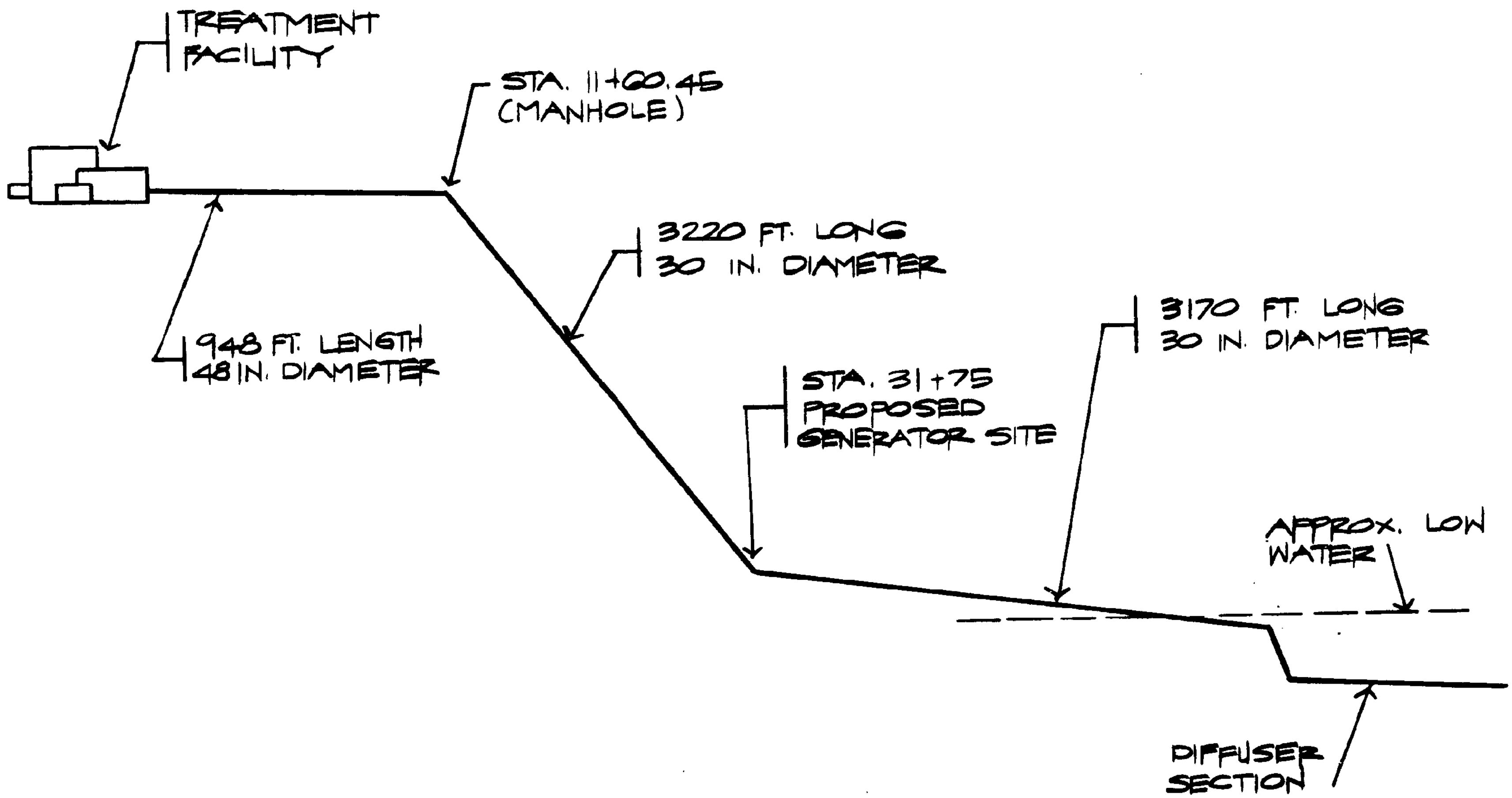

FIGURE 2

NDSS OUTFALL LINE

pressure pipe. The interior walls are extremely smooth, as characterized by the Manning flow coefficient n = 0.0095, Hazen-Williams flow coefficient C=145, or Darcy-Weisbach flow coefficient f = 0.0160. Pipe sections are 20 ft. long and are sealed at bell and spigot type joints by a round "O" ring elastomeric gasket (Refer to Appendix IV for more complete technical information).

2.2 THE WASTEWATER

Raw sewage enters the treatment plant where it is separated, settled, aerated, chlorinated and then discharged. The discharged wastewater has a specific gravity of 1.000474. There are small amounts of solids remaining after the treatment; however, particle size is small and presents no engineering design or technical problem with respect to pump operation. Total solids in the discharge are measured at 480 parts per million (ppm). Filterable solids (with diameter of 40 microns or more) are 40 ppm and dissolved solids (with a diameter of less than 40 microns) are 440 ppm. The composition of solids contained in the discharge is chlorides (135.0 ppm), sodium (87.75 ppm), and others (PO_4, N, Ca, Mg) (117.25 ppm). pH of the water is measured at 7.6.[1]

[1]All analytical information contained in this paragraph derives from Guam Environmental Protection Agency analyses.

2.3 WASTEWATER FLOW

Wastewater enters the Northern District Treatment Facility through two trunk lines. The first, called the Northern Link, originates at Andersen Air Force Base and traverses a semicircular route paralleling Route 9. The second, called the Southern Link, originates in Dededo and services activities along a reasonably direct route to the pump station.

Flow data is derived from measurements of these two inputs to the treatment facility. The measurements themselves are accomplished manually by facility personnel and consist of 12 instantaneous measurements of fluid heights in the two trunk lines taken at 2 hour intervals throughout the day. By tables, the heights are converted to instantaneous flow rates and the sum of the two readings is the instantaneous facility input. The daily average flow at the time of this report is approximately 2.2 million gallons per day. Flow from the Northern Link trunk is approximately twice that of the Southern Link.

Such factors as variation in expertise of staff, measurement techniques and the apocryphal nature of the conversion tables make the precision of the measurements somewhat suspect. The values of flow into the facility are treated circumspectly in this report.

Flow records exist from September 1979 to the present (January 1981). These records show a long term trend of increasing daily volumes. Presently, the volume treated is approximately 2.2 million gallons per day (mgd). Flow data is contained in Appendix B.

The flow data is processed statistically to determine hourly, daily and monthly means. In any given month, the volumetric flow rate at a given hour varies appreciably; the standard deviation in these measurements is about 0.4 mgd. The monthly averages, however, are consistent and a graph of one recent month's daily flow pattern is similar to that of any other. There is no appreciable correlation between wet or dry climatic conditions and wastewater flow.

The flow data is measurement of inputs to the facility. The facility is operating well below its 12 mgd design capacity and it is unlikely that capacity will be reached in the foreseeable future. This implies that the facility can be operated as a reservoir and buffer which can attenuate hourly variations in input; the discharge rate can be rendered almost constant and daily variations can be ignored in the process of this study. Because of the wide difference between capacity and use, even unusual surges can be handled with no difficulty. Such a property of the facility makes the power station more attractive in spite of its unusually small scale, for the design is freed from the constraint of incorporating a complicated and expensive flow regulation system and all the flow can be used for power generation; no overflow need be assumed.

SECTION III

DESIGN CONSIDERATIONS

3.0 THE DISCHARGE LINE

Plan and profile drawings of the discharge line are contained in Appendix C.

The present flow of 2.2 mgd is so small in comparison to the design capacity of 12 mgd that it does not fill the discharge line to any significant elevation above sea level. The design of a generation station must naturally include features which match capacities to flows and permit the cliff-descent segment of the line to remain completely filled. In this way a usable, dynamic pressure head for generation station operation is created.

The generator station is assumed to be located at Station 31 + 75, at an invert elevation of 13.75 feet.[1] This location appears suitable because it is well back from the shore line and at the end of the cliff descent section of the line. The line is buried only six feet beneath the surface at this point and is accessable by excavation.

The cliff descent section ends immediately upstream from the generator station; it begins at station 11 + 60.45, the location of the manhole, at an invert elevation of 267.77 feet. This section is assumed to be completely

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[1]As Built Drawings, Northern District Sewarage System, Austin, Smith and Associates Incorporated, NAVFAC Drawing 73-04-819, dtd 7/23/75.

filled with wastewater. This section of line is capable of holding 15,806 ft^3 or 118,220 gal. of wastewater.

3.1 POTENTIAL PRESSURE VARIATION

In a system which permits average flow of 2.2 mgd (3.404 ft^3/sec), the column of water contained in the cliff descent section can potentially change elevation at a rate of 0.0547 ft. head/sec or 18.28 sec/ft. head. At first glance, this would appear to be unacceptable variation and to imply a requirement for sophisticated water level control in the system.

A solution to this potential problem is at hand and in place, however, in the form of the 947.5 ft. section of 48" pipe delivering wastewater from treatment plant to cliff line. A potential volume of 89,058 gal (11,907 ft^3) is contained in a vertical distance of 7.41 ft. The potential time-rate-of-change of head in this section is only 0.0021 ft/sec or 472.04 sec per ft. head. At the average flow rate, over 58 minutes is required to exhaust the volume contained in this section and change the overall head 7.41 ft or 2.92 %. Such regulation to maintain a partially filled condition in this 48" section is well within the capabilities of the existing system, and a gross regulated head of approximately 258 ft. can be assumed to be routinely available for power generation.

Although discharge of wastewater occurs in the open sea at a depth which varies with the tide, such tidal variations do not influence the head available for power generation. Head is a function of the height of the column of water in the discharge line leading to the proposed generation site (and of flow characteristics, involving fluid-pipe interactions).

3.2 NET HEAD

The head derived in a change in elevation from the mean elevation of the 48" section of the discharge line to the proposed generating site is 258 ft; this represents the gross head available for electric power generation.

Water velocity in the cliff descent section is calculated to be 0.6935 ft/sec at an average daily volumetric discharge rate of 2.2 mgd. Head loss in this section due to friction along the line is negligible. The combined effects of wall roughness and normal joint losses cause Techite pipe to perform similarly to one with an equivalent sand roughness of 1/5000.[2] Calculation using the Manning equation and n = 0.01 (a high value) gives a frictional head loss of less than 1 ft over the entire cliff descent section of pipe.

A net operating head for power operation is therefore assumed to be 257 ft.

3.3 WATER HAMMER

Water hammer is the phenomenon which occurs when a liquid, flowing in a pipe, is abruptly stopped by the closing of a valve. The result is the

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[2]Engineering Report ER-01019, Revision A, dtd. Dec. 17, 1971. Amoco Reinforced Plastics Company. See Appendix D of this report.

creation of a travelling pressure (and density) wave which oscillates through the pipe system until damped out by friction. Water hammer can produce potentially severe stress and must be considered in any study of a hydraulic system.

Direct engineering remedies to potential water hammer conditions exist and present no real problem to the designer. Surge tanks and bypass valves are the two most common solutions.

Water hammer analyses of hydroelectric turbine systems usually take the compressibility of water and the elasticity of pipe walls into account. The resulting equations are quite complex and solutions are normally obtained using computers. In preliminary work such as this, simpler methods following the techniques first developed by Allievi are routinely used. The results of an Allievi analysis indicate a pressure rise of 60 ft (27 lb/in^2) upstream on a two second valve closing in the piping system.

The discharge line uses buried Techite-II pipe 2.5 ft in diameter. Specifications for this pipe state the minimum design pressure as 150 lb/in^2. A nominal 250 ft. head and 60 ft. water hammer over-pressure produces a total pressure of 140 lb/in^2. Since valves which by design limit closing times are commercially available, water hammer does not appear to be a substantial design problem.

3.4 AIR BINDING

An air pocket in a water line can reduce effective pressure and create unacceptable fluctuations in that pressure. The Piping Handbook[3] suggests that in sharp downward slopes, air pockets may form along the top of the pipe at several points and flows between 7 and 10 ft/sec may be required to assure air free conditions. The projected water velocity of 0.69 ft/sec is so low, however, that the possibility that air binding and resultant pressure losses will occur in the proposed system is negligible.

3.5 STATION CIVIL AND MECHANICAL DESIGN

So as to preserve the largest working head possible at the generating station, it is envisioned that pump/turbine intercept of the wastewater flow will occur at the level of the buried pipe at station 31 + 75, at an invert elevation of 13.75 ft. Since the pipe at this station is buried some 6 feet beneath the surface, excavation will be required and a subsurface generating room must be constructed.

Although there is no record of wave action reaching the proposed generation station location, station design must incorporate drainage and protective features to minimize any potential hazard to equipment resulting from the accumulation of water in the generating room, whatever might be its origin.

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[3] Piping Handbook, 5th Edition, McGraw Hill Book Co.

The wastewater flow pattern through the generating station is envisioned as a system of three parallel lines, two of which are designed to accomodate pump-generators to produce power; the remaining line is a by-pass. All lines will be properly valved, the two pump-generator lines may be valved both upstream and downstream from the pumps . Upstream valves on these lines are assumed to be capable of fine adjustment for flow regulation.

In its initial, start up configuration the generating station will contain only one pump-generator. A second parallel pump-generator line is included in the design to account for plant expansion and increased wastewater flow above the present 2.2 mgd. Wastewater flows will increase with future connections to the Northern and Southern links and because of cut-overs in which other portions of the island's sewage system is deverted to the Northern District.

For the purposes of cost estimation in this study, two pump-generator lines are assumed to be of equal size; however, it is desirable to consider dissimilar sized lines and pump-generators during the actual A/E design phase: two dissimilar sized pump-generators may be operated to account more fully for routine daily variations in flow and thus lessen the requirement for flow regulation at the treatment facility. This factor will be important in the future when there are larger routine daily inputs to the plant.

Although increased flow can be expected in the future, no reliable estimates can be made of the time span over which such increases will occur. Although new connections will occur and past trends in new connections can be extrapolated, this will not be useful in light of cut-overs (rerouting of other componants of the island's wastewater flow to the Northern District) which will probably also occur; changes in flow resulting from cutovers will mask any new connection trend. At this time, no schedule of cutovers exists.

It is axiomatic of small hydroelectric power station design that cost effective operation is achieved by automation. On site operators are more expensive than servo mechanisms. Therefore it is assumed that all valving switch gear and controls will, to as great an extent as possible, be automated.

3.6 PUMP-GENERATOR CONFIGURATION AND DESIGN

The static and dynamic constraints of the wastewater discharge system not withstanding, the possible variations in pump-generator configuration and design are almost limitless in their detail. Several possible designs were formulated in the process of this study and cost estimates for these designs were made. The cost estimates were sufficiently uniform across all designs to demonstrate that it is not necessary at this juncture to formulate detailed, equipment-specific designs (two quite practical designs, however, are itemized for future reference in Appendix G).

There are two types of generators: induction or sychronous. Induction generators achieve phase and frequency sychronization by use of an external signal; sychronous generators achieve this sychronization through an independent regulator which is part of the generator controls.
Induction generators are less expensive (by some 10%) because frequency and phase regulation derives from the distribution network to which the generated

elecrical power is supplied rather than specific controls and sensing mechanisms.

The choice of generator type is determined by the use of the generated power: if power is supplied to an existing electrical distribution network, induction generators, being less expensive, are the logical choice. Should it be decided that the generated power will be supplied to the treatment facility for use without comingling with GPA service, synchronous generators are called for. The economics of these considerations (and others) are quantified in Section IV of this report.

An induction generator supplying electricity to the Tanguission entrance to the Guam electrical grid possesses design, operational, and economic benefits far superior to any other combination of generator type and power user. The simplicity of the design tends to insure reliability; control and protective devices required to protect the system in case of malfunction are less elaborate; the existing grid can easily accept all the power generated, regardless of the degree of future expansion of the generating station.

This last consideration is more important than it might first appear: future expansion of the generator system may result in production capacity in excess of treatment station demand. No such limitation exists in connecting the generator system to the GPA distribution grid. The grid can absorb all the power generated, no matter what that quantity might be.

Particulate matter is present in the wastewater discharge (Section II); the chemicals present imply some slight chance for interaction and deterioration of exposed pump surfaces. The turbine system used must be designed for sewage pumping service.

3.7 DIFFUSION

The placement of electrical generation equipment implies, when compared with the original design parameters, a diminished head in the diffusion section of the discharge line and altered characteristics of disbursement of wastewater into the open ocean. The plant presently operates well below design capacity; diffusion occurs under the influence of diminished head at the present time with no adverse affects; thus, such a situation does not appear to be a matter of serious concern. However, this aspect of the design must be carefully examined during the design phase and it may be necessary to open more ports in the diffusion section of the line in order to limit wastewater back up and avoid depreciated turbine performance.

SECTION IV

PERFORMANCE AND ECONOMIC

ANALYSES

4.0 PERFORMANCE ESTIMATE

Any mass elevated with respect to some reference level possesses energy. The fundamental equation is

$$P.E. = mgh, \tag{4.1}$$

where P.E. is the potential energy, m is the mass, g the acceleration due to gravity, and h the vertical elevation above the reference level. Beginning with this equation, and incorporating the definition of power as work per unit time, the basic equation expressing the power potential of water flowing under pressure can be derived and represented as:

$$P = \frac{Qvhe}{550}, \tag{4.2}$$

where P is power (horsepower), Q is flow (ft^3 per second), v is the density (lb per ft^3), h is the head (ft), and e is the efficiency of the conversion system. From the data of preceeding sections, and the assumption of an overall efficiency of 0.7, we have:

$$P = \frac{\dfrac{3.404\ ft^3}{sec.} \times \dfrac{lb}{0.01602\ ft^3} \times 1.000474 \times (254\ ft) \times (0.7)}{550\ \dfrac{ft\text{-}lb/sec.}{horsepower}}$$

$$= 68.72\ hp.$$

Using the theoretical conversion factor from horsepower to kilowatts, we have

$$P = (68.72\ hp) \times (0.7457\ \frac{KW}{hp}) = 51.25\ KW.$$

This calculation is tentative since it assumes without detailed investigation that the overall efficiency of the system is 0.7; however, it is useful in establishing a working estimate of system's economic performance.

Reference to performance data of suitably sized pumps used as turbines (for example, the Bingham-Willamette Company model 4 x 6 x 7 1/2H HTCAP 1-STAGE) indicates a turbine operating efficiency of 78% can be expected. This efficiency, when combined with a generator efficiency of 90% (a typical value), produces an overall system efficiency of 70%. Other pumps used as turbines will yield similar values. Private communication from institutional, contractural, and manufacturing representatives qualified in such applications bear out the assumption that 70% is indeed a reliable working estimate of system efficiency.

4.1 ECONOMIC VALUE OF POWER

Given a generator output of 50 KW, electric power production at present (January 1981) rates represents an annual value of

$$V = (50.0\ \text{KW}) \times (24\ \frac{\text{hr}}{\text{day}}) \times 365\ \frac{\text{day}}{\text{yr}} \times (\frac{\$0.10}{\text{KWH}})$$

$$= \$43,800/\text{year}$$

It can reasonably be assumed that local electrical rates, and hence the value of the power generated, will rise as a function of world oil prices.

4.2 COST ESTIMATE

The value of electric power produced by such a generating station must be weighed against the cost of implementing such a station. This cost divides into the following general catagories:

4.2.1 Turbine/Generator Equipment: The materials cost for turbine, synchronous generator, valving, piping and related electrical controls and relays has been quoted by one contractor experienced in such installations at $15,324 complete (F.O.B. Twin Falls, Idaho). A pump manufacturer has quoted the cost of similar equipment at $12,341 (F.O.B. Schreveport, Louisiana); however, this latter quote omits valving. The higher quote is taken as typical, but is escalated as follows to produce a more conservative cost estimate for materials: assume 20% of materials cost for shipping and 15% increase in materials cost for inflation. A materials cost of

$$M_s = \$15,324 \times 1.15 \times 1.20 = \$21,150$$

is assumed for synchronous generator and

$$M_i = \$13,950 \times 1.15 \times 1.20 = \$19,250$$

for an induction generator.[1]

4.2.2 Civil/Structural Construction: Earthwork necessary to intercept the discharge line and concrete structure required for system protection (24' x 14') are estimated at $27,910. A 20% contingency cost is added. Total civil/structural costs are estimated at $33,500.

4.2.3 Plumbing/Mechanical System Installation: All labor and materials necessary to install the system in place within the protective structure are estimated at $23,000. A 20% contingency overage brings this amount to $27,600.

4.2.4 Transmission: The total installed cost of transmission lines capable of delivering 50 KW at 13000 volts is $1300/180 ft. (this line is capable of handling increased future power transmission

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[1]This price will vary somewhat but induction generator cost will average some 10% below sychronous generators. See Section 3.6 and Appendix G.

resulting from generator plant expansion). Step-up or step-down transformer total cost is $3500 each. Delivery of power to the NDSS treatment plant will involve two transformers and 4200 ft. of transmission line, for a total installed cost of $37,300. This cost is escalated by 20% for contingency to obtain a transmission equipment cost estimate of $44,800.

The generated power may also be fed directly to the Tanguisson Generating Plant for distribution in the GPA net. Such a distribution arrangement requires one step-up transformer and 1600 ft. of transmission line for a total in-place cost of $15,060. A 20% contingency overage brings this total transmission cost estimate to $18,070. There is obvious economic advantage in connection with Tanguisson.

4.2.5 A/E Design, Construction Management and Final Testing: Costs associated with engineering design, construction management, and testing are estimated at between eight and eleven percent of total installed material and labor cost. Because of the innovative nature of the project and the probable requirement for off-island consultants, the larger percentage is assumed.

4.2.6 Total Cost: As mentioned at various points in this report, two system configurations are possible: (1) an induction generator providing power to the GPA grid and (2) a synchronous generator providing power to the NDSS treatment plant. The difference in cost in the two configurations centers on the lower cost for the induction generator and the lower cost of delivery of the power to the GPA grid. There quite possibly may be slightly higher annual operating and maintenance costs for the synchronous system.

TABLE 1

GENERATOR STATION COST

Component	System Type	
	Induction	Synchronous
Turbine/Generator	\$19,250	\$21,150
Civil/Structural	\$33,500	\$33,500
Plumbing/Mechanical	\$27,600	\$27,600
Transmission	\$18,070	\$44,800
A/E Design	\$10,830	\$13,976
TOTAL	\$109,250	\$141,026

Obviously, economic analyses comparing the two systems will give results favorable to the induction generator system. In what follows, only the induction system is analyzed.

4.3 ECONOMIC ANALYSIS

4.3.1 Preliminary Analysis: A rudimentary estimate of the economic benefit of the project may be obtained by simply dividing the initial cost by annual savings and thus obtaining an estimated payback period:

$$\text{Simple Payback} = \frac{\$109{,}250}{\$43{,}800/\text{year}} = 2.49 \text{ year.}$$

This simplistic treatment ignores several important factors such as future worth of money, escalation of utility rates (implying an increased value of the power produced), annual operation and maintenance costs, and system lifetime; however, it serves to point imphatically to the inherent potential of this project.

4.3.2 Detailed Analysis: Governmental agencies traditionally evaluate projects in terms of benefit-cost ratio analysis; however, because the using agency, the Public Utility Agency of Guam (PUAG), presently experiences the same severe financial pressures resulting from increased electrical cost as any commercial enterprise, it is appropriate to express the economic merits of this project in a variety of ways including those conventionally reserved for private business.

4.3.2.1 Value of Electricity Produced Over Lifetime: The production is assumed to yield at a monthly income of $3,650 in present dollars. Equipment lifetime is estimated at 15 years; the future value of money is estimated at 12%; conventional electric costs are assumed to rise at an annual rate of 15%. Future increase in production, the result of plant expansion, is ignored.

The present value[1] of all production over the fifteen year life is $839,713.

4.3.2.2 Annual Operation and Maintenance Cost (O & M): O & M costs are estimated at 10% of installed costs annually, or $10,925. If a 10% annually inflation rate is assumed, the present value of O & M costs over equipment life is $129,370.

4.3.2.3 Salvage Value: Assumed negligible for purpose of this study.

4.3.2.4 Savings to Investment Ratio (SIR): Calculated over the lifetime of the project, the savings to investment ratio may be expressed as

[1]Appendix F contains a definition of the terms and discription of the economic analysis used.

$$SIR = \frac{\text{Net Return}}{\text{Initial investment}}$$

$$= \frac{\$839,713 - \$129,370}{\$109,250}$$

$$= 6.50$$

This is quite good. The system pays for its initial investment six times over during its life.

As cut-overs and new connections increase flow, a second generator can be installed. Since the entire installation cost of the second generator involves the pump-generator but no civil, structural, transmission, or design costs, the economic analysis becomes even more favorable. For example, if a second generator, equal in size to the first, is installed after three years and if a 10% annual inflation rate is assumed, the installed cost of the second generator will be $25,620, which is approximately 23% of the cost of the original station. For this investment, we have now doubled production.

4.3.2.5 Benefit/Cost Ratio (B/C):

With the definition

$$B/C = \frac{\text{benefits - disbenefits}}{\text{costs}}$$

and the categorization of initial installation and annual O & M expenses as costs,

$$B/C = \frac{\$839,713}{\$238,620}$$

$$= 3.52.$$

Again, this is quite good.

4.4 OTHER CONSIDERATIONS

One factor of primary importance is that the proposed generation scheme does not depend on imported fossil fuels. Guam has a narrow and somewhat fragile economic base and it is important to the island's economic well-being that money circulates on-island to as great an extent as possible rather than leaving in consequence of the purchase of off-island goods.

Presently, all of Guam's energy derives from imported fossil fuel. In spite of energy conservation efforts, the purchase of fossil fuel represents a substantial drain from the island's economy. Although the proposed project is admittedly small and of negligible consequence in terms of overall energy consumption, it is extremely significant as a precedent-setting act: it will be the first energy producing system of recent times on Guam not dependent on fossil fuel.

SECTION V

IMPLEMENTATION STRATEGIES

5.0 FUNDING

The key act to be performed in implementing this project is, of course, to secure funding. Since it is unlikely that the Public Utility Agency of Guam will fund this project from department resources, funding must probably derive from fedral resources.[1]

The Clean Water Act Amendments of 1977 emphasized "...more use of systems that reclaim and reuse wastewater, eliminate the discharge of pollutants, and allow for a more efficient use of energy and resources."[2] Grants for construction (85% funding) are discussed in 40 CFR Part 35.[3] It appears likely that this project qualifies under these regulations.

Public Law 95-617, Public Utility Regulatory Policies Act of 1978, Title IV, establishes a loan program meant to stimulate development of additional hydroelectric generating capacity by providing low interest loans to defray up to 90% of the cost of feasibility studies, licenses, and approvals.

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[1]It is interesting to speculate on the possibility of private funds being used for this project; e.g., a commercial firm being granted a concession to utilize the wastewater discharge for production of electricity and sale to GPA. Such a scenario falls outside the scope of this present study but should be examined.

[2]Bureau of National Affairs, Policy and Practice Series, Water Pollution Control, section 931:1011 ff, 1979.

[3]Fedral Register, Vol. 45, No. 249, Wed. Dec. 24,1980.

Although this present project does not directly qualify, its innovative nature and good economic performance may qualify it for a variance.

The U.S. Department of Energy has, in recent years, coordinated the efforts of programs and several agencies with the intent of promoting accelerated development of hydroelectric generation capacity. DOE must necessarily be contacted regarding programs for which this present project qualifies.

5.1 IMPLEMENTATION TASKS

This feasibility study has been conducted with the cooperation, but not the detailed participation, of PUAG. The first task directed at implementation must be to affect liaison with PUAG, to advise them of the consequenses of this study, and to enlist their cooperation. Many decisions concerning this generating station can only be made with PUAG cooperation: the manner of funding construction, the designation of the operator of the station (PUAG, GPA, or a private firm acting under contract), the manner in which reimbursement for power is obtained; all these are pivotal decisions.

This study indicates that the distribution of hydroelectricity to the general consumer through the island grid is desirable from both an engineering and economic point of view. It is therefore critical to the successful implementation of the project that GPA concurrence and support be obtained at the outset. There are a number of issues crucial to the project's success which must involve GPA. The most important of these concerns the precedent setting nature of the project. Elsewhere, public utilities have accepted power originating outside their own system, purchased it (or otherwise reimbursed the producer), and distributed it. (The Public Utilities Regulatory Policy Act mandates this procedure for utility agencies; GPA is exempt by virtue of its small size) There is no provision in current GPA regulations or policy for such an operation; certain provisions of GPA Service Rules[4] qualify conditions under which interconnection can be made.

It should also be noted in this regard the GPA presently is faced with a fixed expense level and declining consumption; in such circumstances, GPA may not be overly enthusiastic about purchasing additional power. One possible solution here might be for GPA to own and operate the generating station.

On the assumption that funding is secured, the project becomes a straight-forward design and construction activity.

This feasibiliy investigation has not uncovered any significant design or construction problems which might arise during the course of implimentation. Quite the contrary, the existing outfall system appears ready-made for the incorporation of a hydroelectric generating plant. Implimentation tasks then become the sequence of A/E review and selection, fee negotiation, A/E design and submittal, contractor selection, construction, and finally start-up and operation.

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[4]Guam Power Authority Service Rules dated October 16, 1979, p. 10.

5.2 DETAILS

Certain specific points, primarily relating to design, must be explicitly treated:

5.2.1 Cutover Plans: A concerted effort must be made to define the plans of the Public Utility Agency of Guam for future cutovers to the NDSS, for this will influence plant design as it pertains to future expansion.

5.2.2 Sizing of Future Generators: As the NDSS expands and flows increase, more pump-generator sets should be brought on line. As flows increase, the treatment facility cannot be so extensively used for flow regulation (although in its normal operation it will continue to serve as a buffer of the wastewater flow). This implies that future pump-generator sets must be sized in order to utilize daily flows as completely as possible, given its natural daily cyclic variation in flow rates. Such considerations will certainly influence the design of the initial generating station building and the manner and extent of the interception of the buried Techite pipe.

5.2.3 Dynamic Load Testing of the Techite Pipe: A preliminary dynamic test of the Techite pressure pipe of the cliff descent portion of the outfall line must be made very early in the project to insure its capacity to function under 250 ft (and more) of head. Leaks (if any) must be identified and repaired.

5.2.4 Coordination with GPA: Liaison must be established with the Guam Power Authority to insure cooperative participation in this project. Engineering coordination must be affected to avoid problems arising from connection of the generating station to the grid.

SECTION VI

CONCLUSION AND RECOMMENDATIONS

6.0 CONCLUSION

From an engineering and economic point of view, this project is feasible.

6.1 RECOMMENDATIONS

Work should immediately commence to secure funding for design and construction of the project. The project should be configured as an induction generator system connected to the GPA power distribution grid. The project should be given high priority for implimentation because of its precedent-setting nature. Personnel familiar with the reverse pump mode of hydroelectric power generation must be included in the design and construction team.

APPENDIX A

GUAM WASTEWATER DISCHARGE DATA

Guam discharges wastewater into the Philippine Sea at four locations along it's western shoreline: The Northern District Sewage System Outfall, Agana Sewage Treatment Plant Outfall, the U.S. Navy Outfall (Apra Harbor), and the Agat Sewage Treatment Plant Outfall.

It was the original intent of this report to investigate the potential for hydroelectric power generation of all Guam outfalls; however, repeated inquiries to Director of Water and Sewage Division, U.S. Navy Public Works Center, failed to produce any data concerning head or volumetric flow of the U.S. Navy Outfall. That outfall is therefore not a part of this study.

The Agana STP Outfall processess the largest volume of wastewater, an average of 6.57 million gallons daily (Table A.1). Next in volume is the NDSS with 2.2 mgd (Appendix B) and last, the Agat STP with 1.3 mgd (Table A.2). Unfortunately, Agana and Agat outfalls discharge their wastewater at heads of no more than twenty feet, making hydroelectric generation impractical. The Agana outfall could deliver only 16 horsepower or 12 KW, the Agat outfall could deliver less than 4 horsepower or less than 3 KW. As shown in Section IV, the NDSS can deliver 50 KW.

TABLE A.1

AGANA STP WASTEWATER FLOW[1]

MONTH (1980)	AVE. DAILY VOLUME (millions gallons)
Jan	5.73
Feb	6.57
Mar	5.8
Apr	5.86
May	6.8
Jun	6.69
Jul	6.74
Aug	6.7
Sep	7.61
Oct	7.44
Nov	6.38
Mean	6.57

[1]Public Utility Agency of Guam records.

TABLE A.2

AGAT STP WASTEWATER FLOW[1]

MONTH (1980)		AVE. DAILY VOLUME (millions gallons)
May		1.5
Jun		0.81
Jul		1.32
Aug		1.07
Sep		1.68
Oct		1.54
Nov		1.20
	Mean	1.30

[1]Public Utility Agency of Guam records.

APPENDIX B

FLOW DATA, NORTHERN DISTRICT SEWAGE SYSTEM

This data is taken from Public Utility Agency of Guam (PUAG) daily records of instantaneous input to the NDSS treatment plant.

Data is analyzed to determine mean daily flow in million gallons per day (mgd). Mean daily flow values appear as the far right hand column of the table. There are occasional gaps in this data, periods for which no instantaneous flow readings were made. When such gaps were encountered in the data, no daily mean was computed. Data was also averaged to determine the mean instantaneous flow at 2, 4, 6, ... hours (24 hour clock time) for each month. These means appear as the bottom line of the tables. A mean daily flow (mgd) for the month was determined. This value appears at the lower right hand corner of the tabulated data.

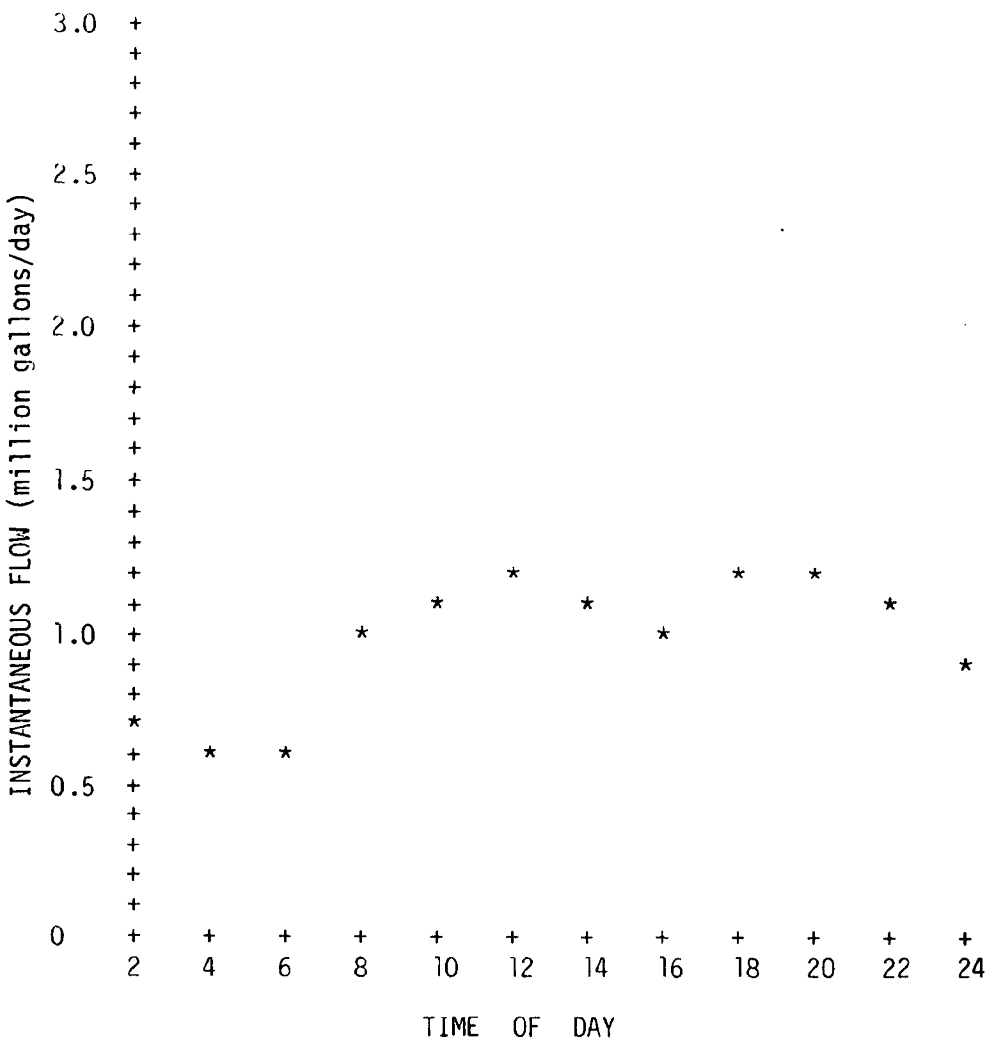
Mean instantaneous flow for 2, 4, 6, ... hours (24 hour clock time) were plotted and appear as graphs on pages immediately following each month's tabulated data.

NDSS Treatment Plant
Parshall Flume Data Table

For: September 1979

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1													
2													
3													
4													
5													
6													
7													
8													
9													
10													
11													
12													
13													
14				1.0	0.9	1.25	0.75	0.9	1.05	1.15	0.82	0.65	
15	0.65	0.4	0.4	0.65	0.82	1.25	1.25	1.07	1.07	1.07	1.07	0.65	0.9
16	0.4	0.4	0.5	0.82	1.25	0.82	0.82	0.82	0.82	0.9	0.9	0.65	0.8
17	0.65	0.65	0.4	0.7	1.2	1.0	1.0	1.2	1.4	1.2	1.0	0.9	0.9
18	0.8	0.5	0.6	1.4	1.4	1.4	1.2	1.2	1.3	1.25	1.25	0.9	1.1
19	0.65	0.8	0.75	0.65	1.25	1.4	1.4	1.2	0.9	1.05	0.9	0.75	1.0
20	0.65	0.65	0.65	1.25	1.25	1.4	1.25	1.25	1.25	1.25	1.25	1.05	1.1
21	0.65	0.4	0.6	1.1	1.25	0.9	0.8	0.75	1.1	1.1	1.25	0.9	0.9
22	0.5	0.5	0.65	1.25	1.25	1.6	1.6	1.25	1.25	1.25	1.1	0.75	1.1
23	0.75	0.5	0.4	1.2	1.3	1.45	1.25	0.9	1.25	1.25	0.75	0.9	1.0
24	0.65	0.65	0.65	0.8	1.05	1.05	1.05	0.9	1.2	1.25	1.25	1.1	1.0
25	1.0	0.65	1.0	1.1	0.9	1.05	1.05	1.05	1.4	1.4	1.4	1.1	1.1
26	0.65	0.65	0.65	1.25	1.05	1.45	0.75	0.9	1.4	1.0	0.75	1.1	1.0
27	0.8	0.8	1.0	1.1	0.9	1.05	0.65	1.1	1.25	1.05	1.25	1.25	1.0
28	0.75	0.65	0.65	0.9	1.05	0.9	0.75	0.65	1.1	1.25	1.05	1.25	0.9
29	0.9	0.65	0.5	0.9	1.45	1.45	1.45	1.45	1.6	1.2	0.9	0.9	1.1
30	0.4	0.65	0.9	0.65	1.25	1.25	1.25	1.25	1.25	1.8	1.25	0.75	1.1
31													
AVG.	0.7	0.6	0.6	1.0	1.1	1.2	1.1	1.0	1.2	1.2	1.1	0.9	1.0

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: September 1979

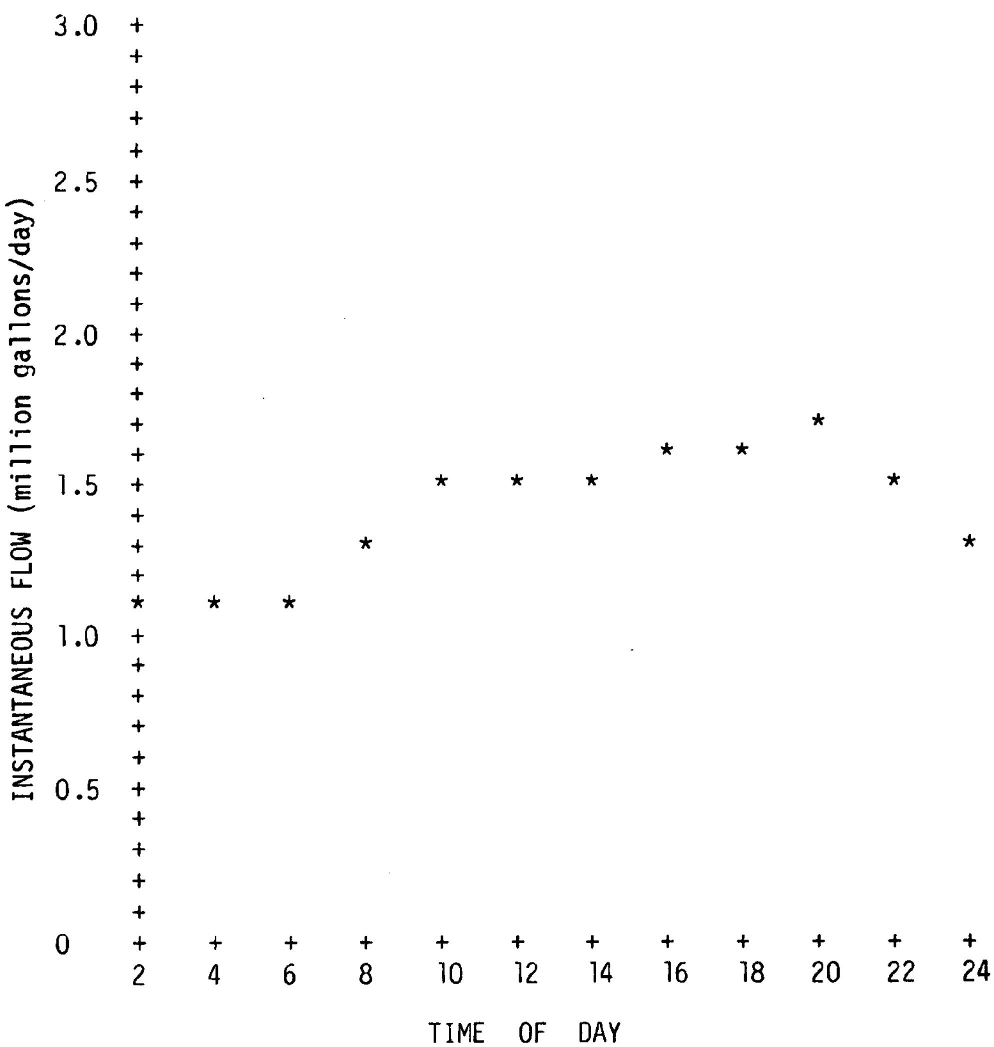


NDSS Treatment Plant
Parshall Flume Data Table

For: October 1979

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	0.75	0.75	0.9	1.25	1.25	1.4	2.0	2.7	2.7	2.1	1.25	1.25	1.5
2	1.25	1.05	1.05	1.25	1.25	1.4	1.25	1.2	1.6	2.0	1.8	1.25	1.4
3	1.25	1.05	1.2	1.4	1.25	1.4	1.25	1.6	1.4	1.4	1.45		
4				0.9	1.4	1.2	1.2	1.4	1.8	1.8	1.6	1.6	
5	1.4	4.7	2.4	1.6	1.4	1.4	1.4	1.6	1.8	2.0	2.0	1.45	1.9
6	0.8	0.75	0.75	0.9	1.6	1.8	1.8	1.6	1.8	1.8	2.0	1.4	1.4
7	0.9	0.75	0.75	1.25	1.8	1.8	2.0	1.6	1.4	1.8	1.8	1.6	1.5
8	1.05	0.9	0.75	0.9	1.25	2.0	1.8	1.4	1.4	1.8	1.6	1.6	1.4
9	1.6	1.05	0.75										
10													
11	1.6	2.0	2.0	1.8	1.8	1.6	1.6	1.8	1.6	1.6	1.6	1.8	1.7
12	1.6	1.6	1.45	2.0	1.8	1.6	1.25	1.25	1.25	1.25	1.25	1.25	1.5
13	1.25	0.9	0.5	2.0	1.8	1.6	1.6	1.4	2.0	2.0	2.0	2.0	1.6
14	1.8	1.05	3.2	1.45	1.6	1.6	1.6	1.6	1.65	1.45	1.05	1.25	1.6
15	1.2	0.9	0.9	2.0			1.4	1.25	1.27	1.6	1.25	1.4	
16	1.05	1.05	1.05	1.6	1.6	1.4	1.4	1.4	1.2	1.8	1.6	1.6	1.4
17	1.6	1.6	1.05	1.2	1.6	1.5	1.6	1.32	1.4	1.6	1.6	1.0	1.4
18	0.7	0.7	1.0	1.2	1.6	1.8	1.4	1.6	1.4	1.2	1.1	0.6	1.2
19	0.6	0.6	0.8					1.1	1.1	1.45	1.25	0.8	
20	0.8	0.65	0.65	1.25	1.2	1.4	1.15	1.25	1.25	1.25	1.45	1.1	1.1
21	0.75	0.75	0.75	1.4	1.4	1.4	1.4	1.45	1.45	1.45	1.05	0.9	1.2
22	0.75	0.75	0.75	0.65	0.75	1.25	1.6	1.8	1.6	1.8	1.45	1.05	1.2
23	1.25	0.9	0.75	1.1	1.05	1.4	1.6	1.8	1.8	1.45	1.45	1.05	1.3
24	0.85	0.75	0.9	1.1	1.05	1.25	1.4	1.4	1.2	1.6	2.0	1.3	1.2
25	0.9	0.85	1.25	1.2	1.6	1.6	1.4	1.6	1.6	1.4	1.25	1.25	1.3
26	1.25	0.9	1.25	1.05	1.3	1.05	1.05						
27				1.25	1.4	1.8	1.4	2.0	1.8	1.6	1.45	1.4	
28	1.4	1.4	1.6	1.1	1.45	1.6	1.8	2.0	2.0	2.0	1.6	1.05	1.6
29	0.9	0.9	1.4	1.4	1.4	1.4	1.6			2.0	1.8	1.6	
30	1.6	0.9	0.9	1.25	2.0	2.0	1.2	1.4	1.6	1.6	1.6	1.25	1.4
31	0.9	0.5	0.75	1.4	1.8	1.6	1.6	2.0	1.6	1.4	1.6	1.6	1.4
AVG.	1.1	1.1	1.1	1.3	1.5	1.5	1.5	1.6	1.6	1.7	1.5	1.3	1.4

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: October 1979

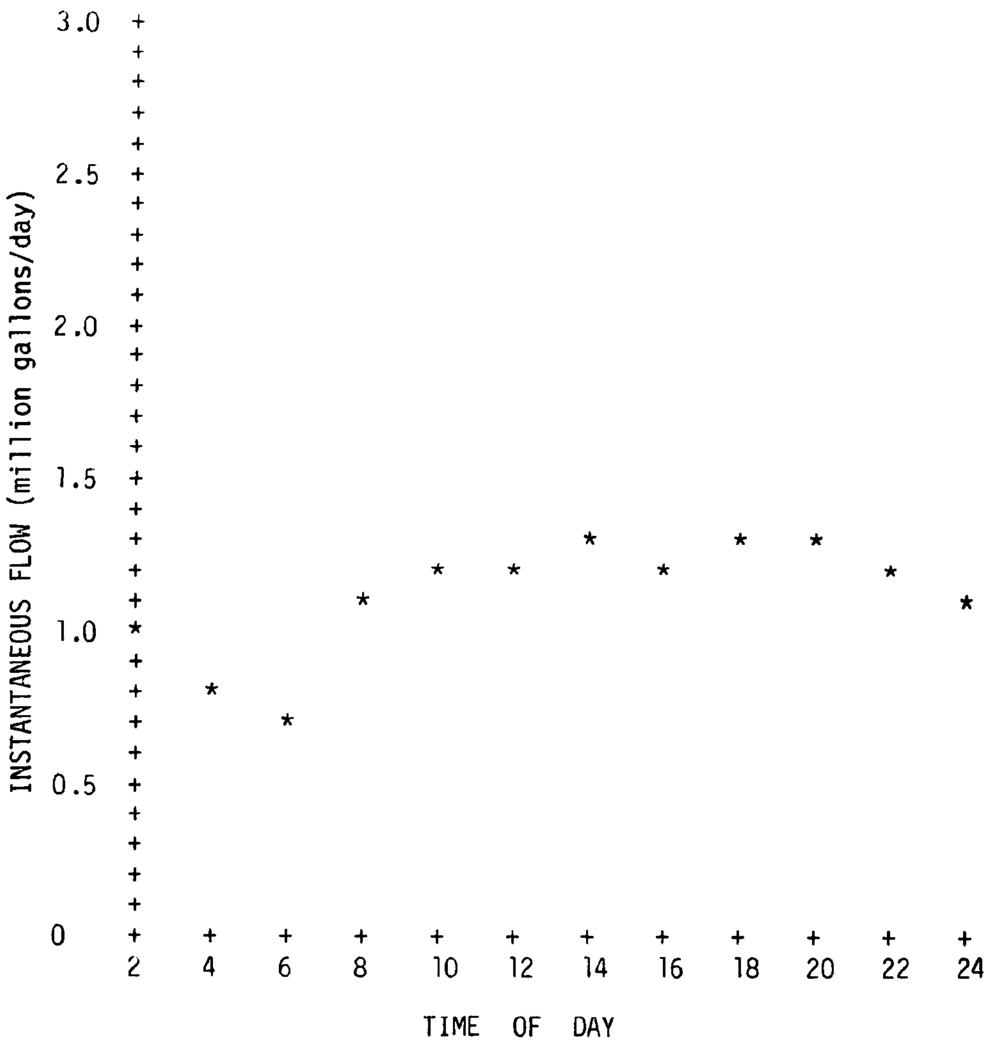


NDSS Treatment Plant
Parshall Flume Data Table

For: November 1979

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.05	0.65	0.75	1.25	1.25	1.4	1.4	1.25	1.4	1.45	1.45	1.45	1.2
2	1.1	1.05	0.9	1.25	1.6	1.8	1.4	1.6	1.45	1.25	0.9	1.25	1.3
3	1.25	0.9	0.9	1.2	1.4	1.4	1.4	1.45	1.2	1.05	0.9	0.9	1.2
4	0.9	0.9	0.6	0.9	1.05	1.2	1.05	1.2	1.2	1.6	1.8	1.25	1.1
5	0.9	1.1	0.65	0.9	0.75	1.3	3.5	2.85	1.8	1.6	1.6	1.6	1.5
6	1.2	1.05	0.9	1.05	1.6	1.8	1.6	1.8	1.8	1.6	1.6	1.8	1.5
7	1.4	0.9	1.05	1.4	1.6	1.8	1.6	1.6	1.6	2.0	2.0	1.6	1.5
8	1.6	1.05	0.9	1.6	1.8	1.8	1.6	1.4	1.6	1.8	1.4	2.1	1.6
9	1.4	1.4	1.05	1.9	2.1	1.8	1.4	1.25	1.4	1.6	1.6	0.75	1.5
10	0.75	0.75	0.65	1.1	1.05	1.4	1.4	1.05	0.9	0.9	0.75	0.75	1.0
11	0.65	0.65	0.65	0.9	1.05	1.05	1.05	1.05	1.05	1.25	1.25	1.05	1.0
12	0.75	0.65	0.65	0.65	0.9	1.05	1.05	1.4	1.6	1.6	1.05	1.05	1.0
13	0.65	0.65	0.65	1.05	1.1	1.25	1.8	0.9	1.05	1.4	1.05	1.05	1.1
14	0.75	0.65	0.65	1.4	1.2	0.75	0.9	1.1	1.25	1.05	0.9	0.9	1.0
15	0.75	0.65	0.65	0.75	0.9	1.05	0.75	0.8	0.9	1.25	1.25	1.4	0.9
16	1.3	0.65	0.65	1.05	1.05	0.9	0.9	0.9	0.9	0.9	1.2		
17				0.75	0.75	0.9	1.05	0.75	1.05	0.9	0.9	1.2	
18	0.9	0.65	0.65	0.8	0.9	1.05	1.05	0.8	1.05	1.05	1.05	0.9	0.9
19	0.65	0.84	0.75	1.05	1.05	1.05	1.1	0.9	1.05	0.9	0.9	0.75	0.9
20	0.75	0.75	0.75	0.8	1.05	1.05	1.25	1.05	1.25	1.25	1.05	0.9	1.0
21	0.9	0.75	0.65	1.3	1.2	1.1	1.2	1.1	1.2	1.05	0.9	0.9	1.0
22	0.75	0.6	0.75	1.0	1.6	1.45	1.6	1.1	1.25	1.05	0.9	0.9	1.1
23	0.75	0.6	0.6	0.8	0.65	0.65	0.65	0.9	1.4	0.9	0.9	0.65	0.8
24	0.65	0.65	0.8	0.9	1.05	1.2	1.4	1.4	1.6	0.9	0.75	0.75	1.0
25	0.75	0.75	0.65	0.65	0.75	1.2	1.25	0.9	0.75	1.15	1.15	1.05	0.9
26	0.75	0.5	0.5	0.9	0.8	0.9	1.2	0.9	0.9	0.9	0.9	0.9	0.8
27	0.9	0.6	0.5	1.4	1.25	0.9	1.25	1.35	1.05	1.65	1.35	1.45	1.1
28	1.45	1.15	0.9	1.2	1.8	1.2	1.05	1.25	1.35	1.8	1.45	1.25	1.3
29	1.6	1.2	0.75	1.4	1.3	1.4	1.05	1.25	1.45	1.9	2.25	1.75	1.4
30	0.5	0.9	0.75	2.0	1.6	1.4	1.05	1.75	1.8	1.4	1.25	1.05	1.3
31													
AVG.	1.0	0.8	0.7	1.1	1.2	1.2	1.3	1.2	1.3	1.3	1.2	1.1	1.1

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: November 1979

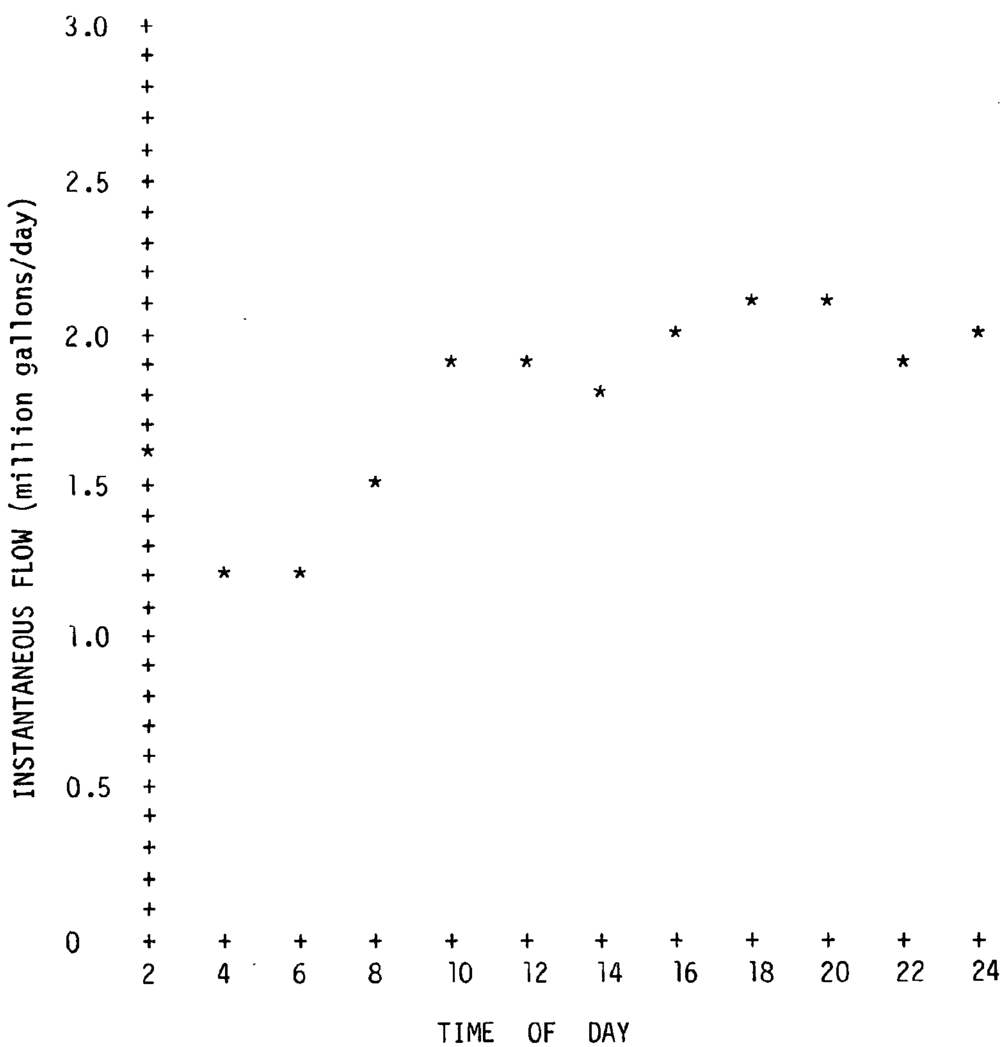


NDSS Treatment Plant
Parshall Flume Data Table

For: December 1979

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.05	0.75	0.6	1.05	1.4	1.6	1.4	1.4	1.6	1.8	1.6	1.4	1.3
2	1.05	1.05	1.4	1.2	2.0	1.6	0.9	1.05	1.4	1.4	1.2	1.4	1.3
3	0.9	0.9	0.65					1.8	2.0	2.0	1.8	1.6	
4	1.6	1.2	1.05					1.6	1.9	1.9	1.6	1.2	
5	1.3	1.05	1.05	1.4	2.4	1.9	2.1	2.1	2.1	1.9	1.8	1.6	1.7
6	1.4	0.9	0.9	1.6	1.8	1.9	1.8					1.6	
7	1.4	1.05	0.9	1.4	2.5	2.1	2.3	1.6	2.1	1.8	1.8	1.8	1.7
8	1.4	1.05	0.9	1.6	2.1	2.4	1.9	1.4	1.9	2.1	2.1	1.8	1.7
9	1.4	1.05	0.9	1.05	1.4	2.1	2.1	1.8	2.0	2.1	1.8	1.8	1.6
10	1.4	1.05	0.9	1.2	1.8	2.3	2.1	1.6	1.8	1.8	2.0	1.45	1.6
11	1.6	1.4	1.65	1.8	1.9	1.9	1.6	1.8	2.3	2.3	2.3	2.4	1.9
12	1.9	1.25	1.3	2.1	2.1	1.9	1.9	1.9	1.8	2.1			
13				1.8	2.1	1.8	2.3	1.8	2.3	1.8	1.9	1.9	
14	1.8	1.05	1.25	2.5	2.6	2.4	2.3	2.3	2.1	2.3	2.1	2.3	2.1
15	1.6	1.25	1.6	1.2	1.6	1.8	1.8	1.8	2.3	2.2	1.7	2.6	1.8
16	1.9	1.45	1.45	1.2	1.6	1.9	1.8	1.8	2.1	2.3	1.8	2.0	1.8
17	1.5	1.4	1.05	1.2	1.4	1.4	1.8	1.8	2.3	2.2	1.45	2.6	1.7
18	2.0	1.25	1.17	1.8	1.8	1.6	1.6	2.0	2.0	1.3	1.05	2.0	1.6
19	1.4	1.4	1.8	1.6	1.6	1.6	1.6	2.6	2.8	2.8	2.1	2.1	2.0
20	1.9	1.45	0.99	1.6	1.6	1.6	1.6	3.0	3.0	2.6	2.8		
21				1.6	1.8	1.8	1.6	2.1	2.5	2.7	3.0	2.8	
22	2.0	1.45	1.25	1.4	1.8	2.0	1.5	2.4	2.4	2.6	1.75	2.1	1.9
23	1.9	1.4	1.4	1.4	2.0	1.8	1.9	2.9	1.8	1.8	2.6	2.6	2.0
24	2.6	2.0	1.2	1.4	1.8	2.0	1.8	2.5	2.45	2.8	2.6	2.0	2.1
25	1.8	1.25	1.25	1.2	1.5	1.8	1.6	2.0	2.3	2.0	1.8	2.1	1.7
26	2.0	1.2	1.3	1.05	1.4	1.8	1.6						
27				1.05	1.9	2.1	2.6	2.1	2.0	2.3	2.1	2.3	
28	1.8	1.4	1.2	1.6	2.1	2.1	1.8	2.1	1.8	1.6	1.6	1.6	1.7
29	1.8	1.45	1.05					2.0	2.1	2.1	2.0		
30													
31				2.0	2.1	3.0							
AVG.	1.6	1.2	1.2	1.5	1.9	1.9	1.8	2.0	2.1	2.1	1.9	2.0	1.7

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: December 1979

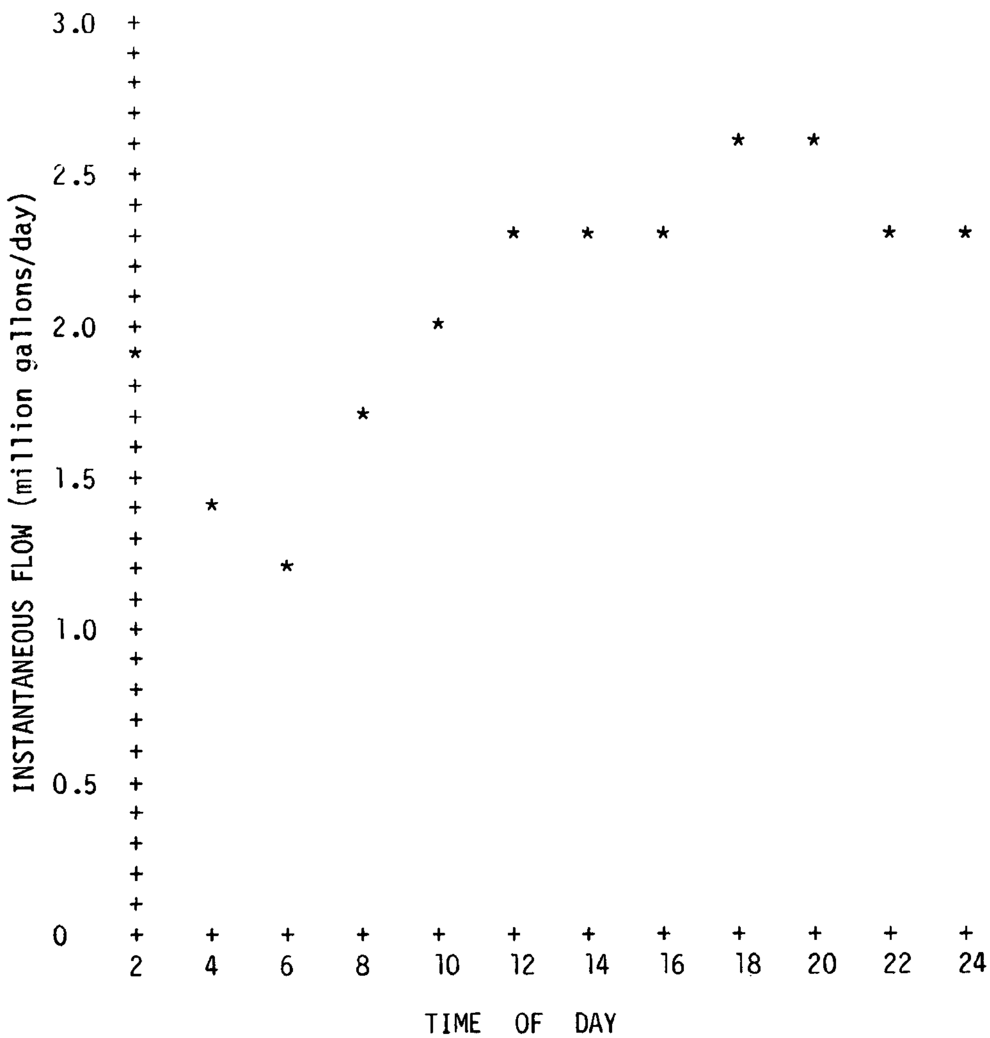


NDSS Treatment Plant
Parshall Flume Data Table

For: January 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1				1.0	1.0	2.3	1.8	2.0	2.8	2.8	2.8	2.1	
2	1.8	0.9	0.9										
3												2.6	
4	2.0	1.6	1.05	1.15	1.9	2.1	1.65						
5													
6												2.6	
7	2.6	1.6	1.4	1.75	2.6	1.9	2.1	2.1	2.8	2.8	2.6	2.0	2.2
8	1.9	1.65	1.8					2.1	2.8	3.0	2.8	2.8	
9	2.8	1.6	1.4	1.8	2.6	2.1	2.6	2.1	2.8	2.8	2.6	2.1	2.3
10	1.6	1.25	0.75	2.1	1.8	2.1	2.6	2.1	2.8	2.8	2.6	2.1	2.1
11	1.6	1.25	1.05	1.8	2.1	2.1	2.6	2.1	3.0	2.8	2.6	2.1	2.1
12	2.2	1.4	0.9	1.6	1.8	2.6	2.7	2.1	2.65	2.8	2.6	2.6	2.2
13	2.1	1.25	1.25	1.1	1.6	2.45	2.8	2.8	2.8	2.6	1.9	2.1	2.1
14	1.7	1.25	1.25	1.5	1.6	2.45	2.8	3.0	3.0	2.8	3.0	2.8	2.3
15	2.1	1.9	0.9	1.8	2.0	2.9	2.8	2.8	3.0	2.8	1.6	2.6	2.3
16	1.75	1.2	1.0	1.65	2.3	2.4	2.6						
17				1.65	1.9	2.8	2.6	2.3	2.3	2.1	1.6		
18				1.65	1.9	2.8	2.6	2.1	2.3	2.1	2.1	2.4	
19	1.45	1.19	0.9	1.65	1.9	2.8	2.6	2.1	2.1	2.0	1.9		
20				1.8	2.3	1.8	1.4	2.1	2.1	1.9	1.9	2.25	
21	1.65	1.25	1.05	2.0	2.9	2.3	2.3	2.8	2.6	2.1	2.3	2.6	
22	2.4	1.6	1.4	1.6	2.0	2.3	2.3	2.1	2.1	2.1	1.9	2.1	
23	1.6	1.4	1.5	1.8	2.0	2.3	2.3	2.3	2.8	2.8	2.6	1.6	
24	1.6	1.2	0.9	1.6	2.3	2.3	2.3	2.1	2.0	2.6	2.6	2.1	
25	1.6	1.6	1.25	1.4	1.8	2.3	2.3	2.7	2.6	2.8	2.8	2.6	
26	1.8	1.2	1.25					2.7	2.7	2.8	2.6	2.8	
27	2.1	2.3	2.8					2.7	2.6	2.3	2.1	2.6	
28	1.4	1.25	0.9	1.4	2.1	2.1	2.3	2.1	2.6	2.6	2.6		
29				2.3	2.1	2.1	2.3	2.1	2.5	2.5	2.5		
30				1.8	2.1	2.1	2.1	2.1	2.8	2.9	2.6		
31				1.9	2.1	2.1	1.9	2.2	2.8	2.4	1.45	1.45	
AVG.	1.9	1.4	1.2	1.7	2.0	2.3	2.3	2.3	2.6	2.6	2.3	2.3	2.2

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: January 1980

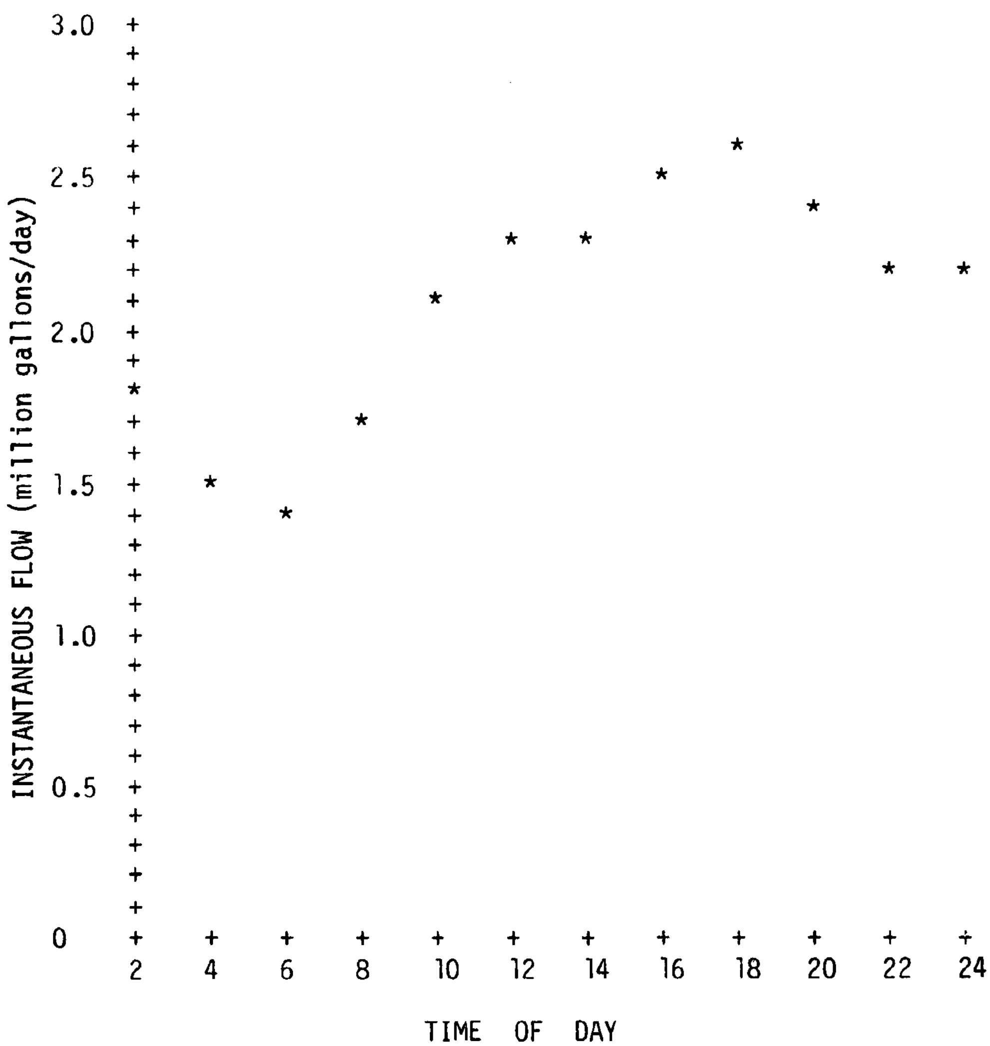


NDSS Treatment Plant
Parshall Flume Data Table

For: February 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.45	0.9	0.75	2.2	2.3	2.1	3.0	2.4	2.5	2.4	2.4		
2													
3													
4				2.1	2.3	2.3	2.3	2.0	2.1	2.1	2.3	2.1	
5	1.6	1.4	1.4	1.9	2.2	2.8	2.4	2.8	2.8	1.8	1.4	2.1	2.1
6	1.9	1.45	1.45	2.0	2.0	3.3	2.7	2.6	2.8	1.8	1.6	1.6	2.1
7	1.6	1.6	1.2	1.05	2.0	2.3	2.4	2.8	2.3	2.3	1.75	2.1	2.0
8	1.8	1.6	1.2	1.4	1.8	2.3	2.2	2.3	2.3	1.6	1.25	1.9	1.8
9	1.9	1.25	1.25	1.4	1.8	2.3	2.3	2.3	2.3	1.6	1.05	1.9	1.8
10	1.65	1.45	1.45	1.4	1.8	2.3	2.5	2.8	2.8	2.1	1.45	1.9	2.0
11	1.45	1.25	1.6	1.6	1.8	2.3	2.6	2.1	2.3	2.3	2.1		
12	2.3	1.8	1.6	1.4	1.8	2.3	2.3	2.1	2.3	2.3	2.1	2.6	2.1
13	1.6	1.2	1.6	1.4	1.8	2.0	2.0	2.1	1.8	1.8	2.3	2.7	1.9
14	2.0	1.6	1.4	2.0	2.3	2.1	2.3	2.2	2.1	2.1	2.1	2.1	2.0
15	1.9	1.45	1.4					2.3	1.9	2.1	2.1	2.7	
16	1.8	1.6	1.05									2.0	
17	1.6	1.6	1.4					2.1	2.1	2.6	2.6	2.1	
18	1.8	1.8	2.1	2.0	2.3	2.3	2.1	2.8	2.8	2.8	2.8	2.3	2.3
19	1.8	1.8	2.1	1.25	2.1	2.1	2.1	2.3	2.8	2.8	2.8	2.1	2.2
20	1.4	1.25	1.25	1.8	2.1	2.1	2.3	2.1	2.6	2.8	2.8	2.1	2.1
21	2.1	1.6	1.25	1.25	2.1	2.1	2.6	2.6	2.8	2.6	2.8	2.8	2.2
22	2.1	1.8	1.4	1.35	2.1	2.6	2.6	2.8	3.2	3.0	2.8		
23				1.8	2.1	2.3	2.1	2.8	2.8	3.2	2.8		
24				1.8	2.3	2.1	2.1	2.1	2.8	2.8	2.6		
25				2.1	2.6	2.8	2.6	2.3	3.0	2.9	2.1		
26								4.5	4.1	2.3	2.6	3.1	
27	1.45	1.25	1.19	1.9	2.6	2.1	2.1	2.6	2.8	3.0	2.1	2.1	2.1
28	1.6	1.6	1.6	1.75	1.9	2.1	2.1	2.6	2.8	2.9	2.1	2.1	2.1
29	1.7	1.25	1.25	1.4	1.8	2.1	2.1	2.3	2.3	2.1	1.8	1.8	1.8
30													
31													
AVG.	1.8	1.5	1.4	1.7	2.1	2.3	2.3	2.5	2.6	2.4	2.2	2.2	2.0

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: February 1980

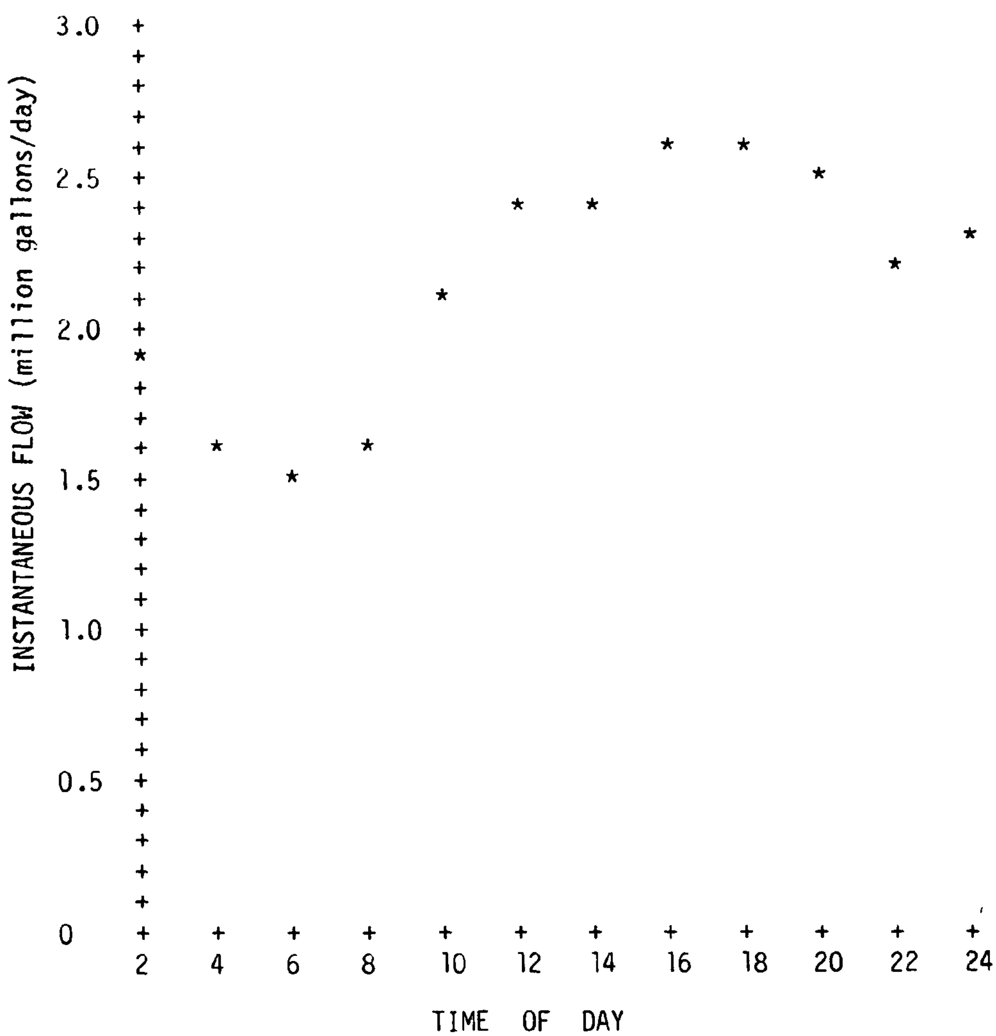


NDSS Treatment Plant
Parshall Flume Data Table

For: March 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.6	1.6	1.4	1.4	2.1	2.1	2.6	2.8	2.1	2.4	1.75	2.1	2.0
2	1.9	1.6	1.4	1.3	2.0	2.8	2.1	2.3	2.3	2.0	1.8	2.1	2.0
3	1.8	1.6	1.4	1.3	2.2	2.3	2.8	2.6	2.4	2.8	2.4	2.25	2.2
4	1.45	1.45	2.4	1.3	2.0	2.3	2.3	2.8	2.8	2.6	2.1	2.3	2.2
5	2.3	1.6	1.6	1.3	2.0	2.3	2.3	2.8	2.1	2.1	1.8	2.6	2.1
6	2.1	1.6	1.9	1.3	2.0	2.3	2.3	2.8	2.1	2.1	2.3		
7				1.05	1.6	2.0		2.1	2.3	2.1	2.1	1.8	
8	1.45	1.25	1.25									2.6	
9	2.6	1.9	1.6										
10													
11													
12				2.1	2.3	1.8	2.3	2.6	1.9	1.75	1.75	1.9	
13	1.75	1.65	1.05	2.3	2.8	2.6	2.3	2.25	2.25	2.6	2.25	2.4	2.2
14	2.1	1.8	1.4	1.6	2.1	2.3	2.1	1.75	2.4	2.8	2.6	2.6	2.1
15	1.6	1.4	1.4	1.45	2.1	2.1	2.3	2.6	2.8	2.8	2.9		
16				1.8	2.1	2.1	2.6						
17													
18				1.8	2.1	2.1	2.1	2.8	2.8	2.9	2.1	2.3	
19	2.6	1.6	1.8	1.6	2.1	2.1	2.1					2.8	
20	2.3	1.45	1.4	2.0	2.1	2.6	2.7	2.6	3.1	3.0	2.3	2.8	2.4
21	2.1	1.6	1.8	2.3	2.1	2.6	2.8	2.6	3.1	2.9	2.1	2.1	2.3
22	2.1	1.8	1.6	1.4	1.6	2.4	2.1	2.1	2.8	2.8	2.6	2.1	2.1
23	1.45	1.25	1.15	2.1				2.6	2.9	3.1	3.1	2.1	
24	2.1	2.1	1.6	2.0	1.9	1.75	2.4	2.7	3.2	3.0	2.6		
25				1.45	1.9	2.6	2.25	2.25	2.8	1.8	2.0	2.6	
26	2.1	1.8	1.35	1.8	2.1	2.3	2.3	2.8	2.8	2.3	2.1	1.8	2.1
27	1.4	1.25	1.25	1.8	2.1	2.8	2.8	3.0	2.8	2.4	2.1	2.4	2.2
28	1.75	1.75	1.6	1.8	2.5	2.5	2.5	2.8	3.0	2.3	2.3	2.6	2.3
29	1.9	1.45	1.25	1.6	2.3	2.8	2.8					2.6	
30	1.9	1.6	1.45	1.4	2.1	2.8	2.8	2.3	2.3	2.5	2.1	2.6	2.2
31	1.9	1.6	1.2	1.4	2.1	2.8	2.8	2.8	2.1	2.1	2.3	1.9	2.1
AVG.	1.9	1.6	1.5	1.6	2.1	2.4	2.4	2.6	2.6	2.5	2.2	2.3	2.2

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: March 1980

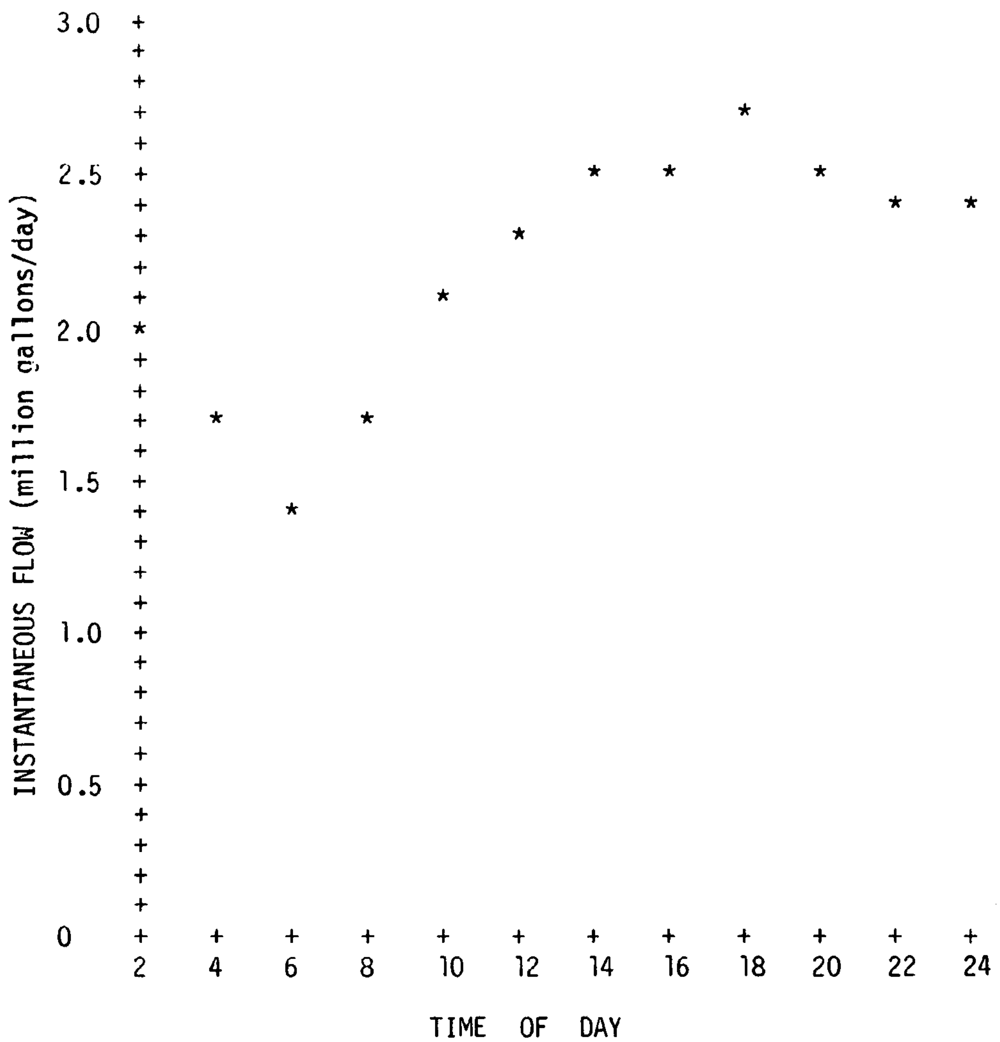


NDSS Treatment Plant
Parshall Flume Data Table

For: April 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.6	1.6	1.4	2.5	2.8	2.8	2.8	2.8	2.8	2.1	2.6	2.6	2.4
2	1.9	1.6	1.4	1.8	2.0	2.3		2.5	2.8	2.2	2.2	2.6	
3	1.9	1.6	1.6	2.2	2.8	2.8	2.8	2.8	2.8	2.2	2.1	2.6	2.4
4	2.6	1.8	1.05	2.0	2.0	2.3	2.0	2.3	2.9	2.8	2.1	2.6	2.2
5	1.9	1.45	1.05	2.3	3.0	3.0	2.8	2.6	2.7	2.7	2.5	1.25	2.3
6	1.25	1.25	1.25	1.8	1.8	1.8	1.6	2.8	2.7	2.9	2.4	2.4	2.0
7	1.75	1.65	1.4	1.4	2.0	2.1	2.6	1.75	2.25	2.6	2.6	2.4	2.0
8	1.9	1.25	1.05	1.8	2.3	2.6	2.6	2.6	3.0	2.6	2.8	2.1	2.2
9	1.8	1.6	1.6	2.3	2.3	2.1		2.25	2.25	2.25	2.45	2.6	
10	2.6	1.9	1.9	2.0	2.3	2.1	2.6					2.8	
11	2.8	2.1	1.9	2.1	2.8	2.1	2.8	2.8	2.9	3.0	2.9	1.6	2.5
12	1.2	1.4	1.05	1.5	2.1	2.1	2.3	2.3	3.0	3.0	2.8	2.8	2.1
13	2.8	2.3	1.6	1.25	1.25			2.8	3.0	2.8	2.8	2.8	
14	2.8	2.3	1.8	1.6	2.25	1.75	2.7	2.8	2.9	3.3	2.8	2.1	2.4
15	1.8	1.4	1.05	2.1	1.75	1.75	1.75	2.6	2.8	2.8	2.8	2.8	2.1
16	2.1	2.1	1.6	1.9	2.4	2.6	3.0	2.8	2.8	2.8	2.8	2.8	2.5
17	2.9	2.1	1.8	1.9	1.45	2.25	2.25	2.4	2.8	3.2	2.8	2.8	2.4
18	2.6	2.1	1.6	1.6	2.45	2.25	2.25	3.0	2.6	2.1	1.9	2.1	2.2
19	2.1	2.1	1.6	1.05	1.6	2.1	2.8	2.8	2.1	1.8		2.3	
20	2.1	2.1	1.6	0.9	1.8	2.3	2.3	2.3	3.1	3.1	1.8	2.25	2.1
21	1.75	1.45	1.2	1.05	2.2	2.3	2.8	2.8	2.8	2.8	2.3	2.45	2.2
22	2.25	1.45	1.2	1.05	1.6	2.1	2.8	2.8	2.1	2.1	2.1	2.4	2.0
23	1.75	1.45	1.4	1.05	2.0	2.3	2.8	2.8	2.6	2.3	2.3	2.4	2.1
24	1.75	1.45	1.4	1.05	1.8	2.1	2.8	2.8	2.6	2.5	2.8	2.4	2.1
25	1.75	1.0	1.25					2.1	2.3	2.3	2.6	1.45	
26	1.45	1.65	0.9	1.8	2.3	2.8	2.3	2.1	2.3	2.6	2.6	2.1	2.1
27	1.6	1.6	2.1	1.8	1.6	2.1	1.6	2.1	2.7	2.2	2.1	2.8	2.0
28	1.9	1.25	0.9	1.8	1.9	2.8	2.8	2.8	2.6	2.3	2.3	2.8	2.2
29	2.3	1.6	1.05	2.3	2.8	2.8		1.9	2.25	2.25	1.75	2.6	
30	2.3	1.4	1.05	1.8	2.3	1.8	2.1	2.0	2.5	2.25	1.75	2.3	2.0
31													
AVG.	2.0	1.7	1.4	1.7	2.1	2.3	2.5	2.5	2.7	2.5	2.4	2.4	2.2

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: April 1980

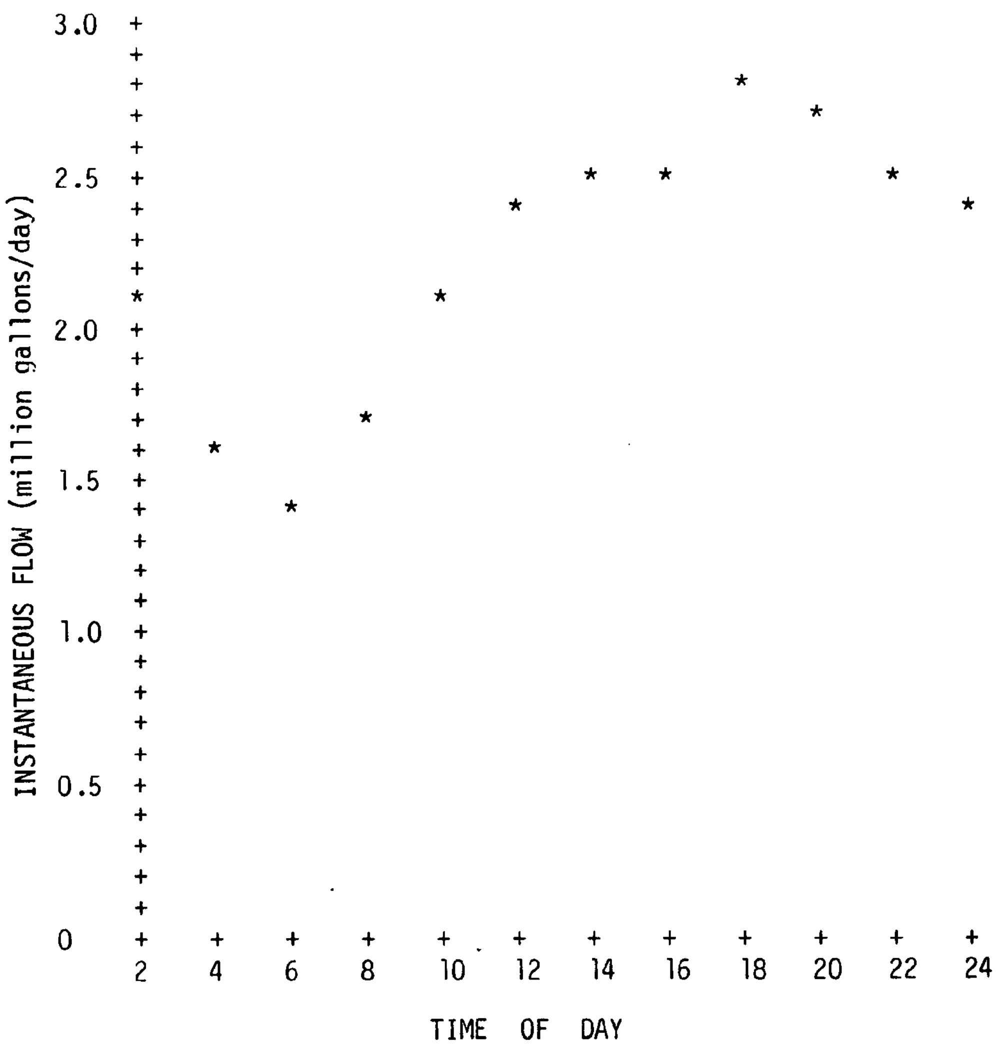


NDSS Treatment Plant
Parshall Flume Data Table

For: May 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.6	1.6	1.05	1.6	2.6	2.1	2.1	1.9	2.4	2.25	1.75	1.75	1.9
2	1.75	1.45	1.05	1.6	2.1	2.3	2.1	2.1	2.6	2.4	1.9	2.12	2.0
3	1.4	1.4	2.0	1.6	2.6	2.8	2.3	2.6	2.4	2.6	2.6	2.6	2.2
4	1.8	1.6	1.4	1.4	2.3	2.3		3.0	2.8	2.6	2.4	2.6	
5	2.1	1.6	1.6	1.4	1.6	2.8	2.3	2.3	2.8	2.8	2.8	2.8	2.2
6	2.3	1.8	1.8	1.5	1.8	2.1	2.1	2.1	2.6	2.8	2.8	2.8	2.2
7	2.8	1.6	0.9	1.8	1.9	2.4	2.4	2.1	2.8	3.0	3.0	2.8	2.3
8	2.8	2.1	1.8	1.6	1.6	2.1	1.9	2.1	2.5	2.1	1.6	2.1	2.0
9	1.8	1.8	1.2	1.6	2.4	2.4	2.4	2.6	2.8	2.8	2.8	2.1	2.2
10	1.8	1.25	1.15	1.25	1.45	1.9	2.6	1.9	2.8	3.1	2.3	1.9	2.0
11	1.6	1.05	0.99	1.25	1.75	2.4	2.6	2.6	2.8	2.3	1.8	2.1	1.9
12	2.1	1.9	1.6	1.75	2.6	2.9	2.9	2.6	2.6	2.1	1.6	2.0	2.2
13	1.4	1.25	0.75	1.05	1.4	2.1	2.1	2.3	2.3	1.8	2.1	2.1	1.7
14	2.1	1.8	1.4	1.4	2.1	2.8	2.8	2.8	3.1	2.8	2.1	1.9	2.3
15	1.75	1.45	1.25	1.4	1.8	2.3	3.0	2.8	2.8	2.6	1.6	2.4	2.1
16	1.75	1.45	1.25	1.5	1.75	2.4	2.25	3.2	3.0	2.6	2.1	1.8	2.1
17	1.9	1.45	2.25	1.3	1.45	1.75	2.6	2.8	2.8	2.9	2.9	2.7	2.2
18	2.1	1.9	1.6	1.25	1.6	1.9	2.6	2.3	2.6	2.3	2.1	2.6	2.1
19	2.4	1.45	1.75	1.6	2.8	2.7	2.4	2.6	3.0	2.7	2.6	2.6	2.4
20	2.4	1.9	1.6	1.8	1.8	2.3	2.3	2.8	3.0	2.9	2.8	2.8	2.4
21	2.1	1.6	1.4	2.0	2.3	2.8	2.8	3.2	3.3	2.8	2.7	2.6	2.5
22	2.1	1.4	1.5	2.0	2.1	2.6	2.8	3.0	3.0	2.7	2.7	2.6	2.4
23	2.3	1.4	1.25	2.3	2.3	2.6	2.4	2.6	2.7	2.7	2.7	1.9	2.3
24	2.4	1.45	1.25										
25				3.0	2.8	2.8	3.1	2.8	3.1	2.9	2.9		
26				2.1	2.3	2.3	2.8	3.2	3.5	4.5	4.9	3.0	
27	3.1	2.8	2.7	1.8	2.1	2.3	2.7	2.6	2.6	2.8	3.0	3.0	2.6
28	2.6	2.1	1.6	2.1	2.9	2.9	2.8	2.1	2.8	2.6	2.6		
29													
30								2.6	2.8	3.2	2.9	2.1	
31	2.3	1.8	1.05	1.4	2.0	2.8	2.6	2.25	2.8	2.4	1.75	2.8	2.2
AVG.	2.1	1.6	1.4	1.7	2.1	2.4	2.5	2.5	2.8	2.7	2.5	2.4	2.3

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: May 1980

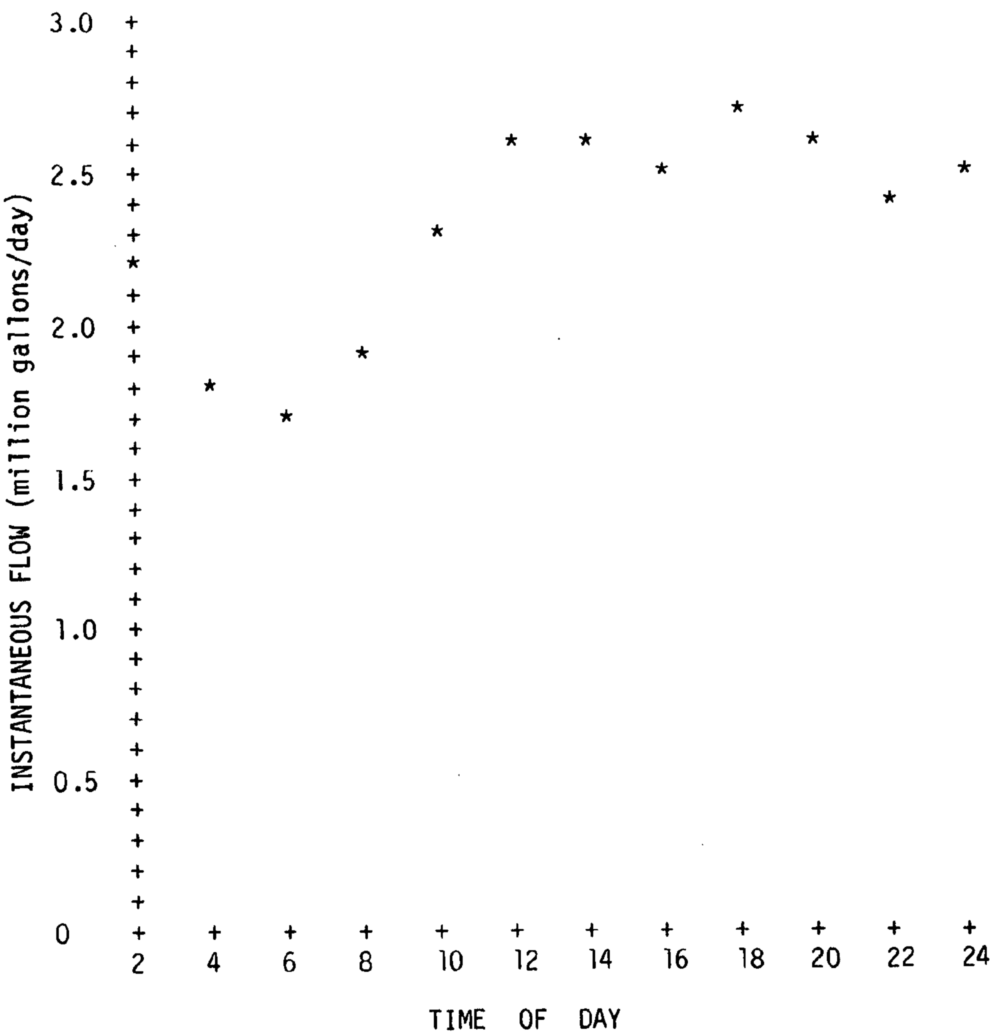


NDSS Treatment Plant
Parshall Flume Data Table

For: June 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	2.6	2.4	1.4	1.4	1.8	2.3	2.6	2.6	4.4	3.3	2.8	2.8	2.5
2	2.2	2.1	1.1	2.1	2.1	2.1	2.6	2.9	2.8	2.1	2.2	2.0	2.2
3	2.0	2.0	1.6	2.1	2.1	2.6	2.1	2.1	2.9	3.1	2.1	2.1	2.2
4	1.9	1.65	1.45	1.8	2.3	2.4	2.7	2.3	2.3	1.8	1.6	2.1	2.0
5	1.9	1.45	1.45	2.1	2.6	2.6	2.5	2.1	2.8	2.1	1.6	2.7	2.2
6	2.4	1.9	1.9	1.9	1.8	2.3	2.3	2.5	2.3	2.3	2.1	2.3	2.2
7	2.1	1.8	1.45	2.0	2.1	3.1	2.8	2.1	2.3	2.3	2.0	1.9	2.2
8	1.6	1.6	2.1	1.4	1.6	1.8	1.8	2.1	2.6	2.4	1.9	2.5	2.0
9	2.25	1.75	2.35	1.6	2.7	3.2	2.6					2.4	
10	1.75	1.45	1.45	1.6	2.0	2.3	2.1	2.8	2.8	2.6	2.1	2.4	2.1
11	1.65	1.45	1.75	1.9	2.1	2.3	2.3	2.3	2.8	2.8	2.8	2.6	2.2
12	1.9	1.75	2.5	2.8	2.8	2.6	2.8					2.1	
13	1.9	1.6	1.75	2.3	2.8	2.8	3.6	3.1	2.8	2.8	2.6	2.6	2.6
14	1.75	1.25	1.6	2.8	2.3	2.1	2.3	2.1	2.8	2.8	3.0	2.3	2.3
15	2.1	1.45	1.25	2.8	2.7	2.2	1.8	2.1	2.6	2.6	2.6	2.8	2.3
16	2.0	1.8	1.6	2.9	2.8	2.6	2.6	2.6	2.6	2.7	2.8	2.8	2.5
17	2.6	2.1	1.8	2.1	2.6	2.8	2.1	2.5	2.6	2.4	2.4	2.4	2.4
18	1.9	1.25	1.45					2.4	2.6	2.6	2.6		
19				1.6	2.1	2.6	2.6	2.6	2.6	2.7	2.6	2.7	
20	2.8	2.1	1.9	1.6	2.1	2.6	2.6	2.4	2.5	2.8		2.8	
21	2.8	2.1	2.1	2.3	2.1	2.8	2.8	2.8	2.7	2.7	2.6	3.0	2.6
22	2.9	2.6	1.9	1.8	2.1	2.3	2.6	2.1	2.6	2.9	3.2	2.6	2.5
23	2.1	2.3	2.3					2.1	2.8	3.1	3.3	2.3	
24	1.8	1.4	1.2	2.0	3.0	3.3	3.5	3.0	2.3	2.9	3.0	2.8	2.5
25	2.8	2.1	2.1	1.4	2.4	2.7	2.8	2.8	3.2	3.2	2.8		
26				1.6	2.6	2.7	3.0						
27				1.4	2.4	2.8	2.8	2.9	3.3	2.8	2.1		
28				1.5	2.1	2.8	2.8	3.0	2.3	2.4	1.9	2.8	
29	2.4	1.9	1.9	1.4	1.6	2.6	2.9	2.8	2.6	2.1	2.1	2.6	2.2
30	2.1	1.9	1.45	2.0	2.1	2.6	2.3					2.2	
31													
AVG.	2.2	1.8	1.7	1.9	2.3	2.6	2.6	2.5	2.7	2.6	2.4	2.5	2.3

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: June 1980

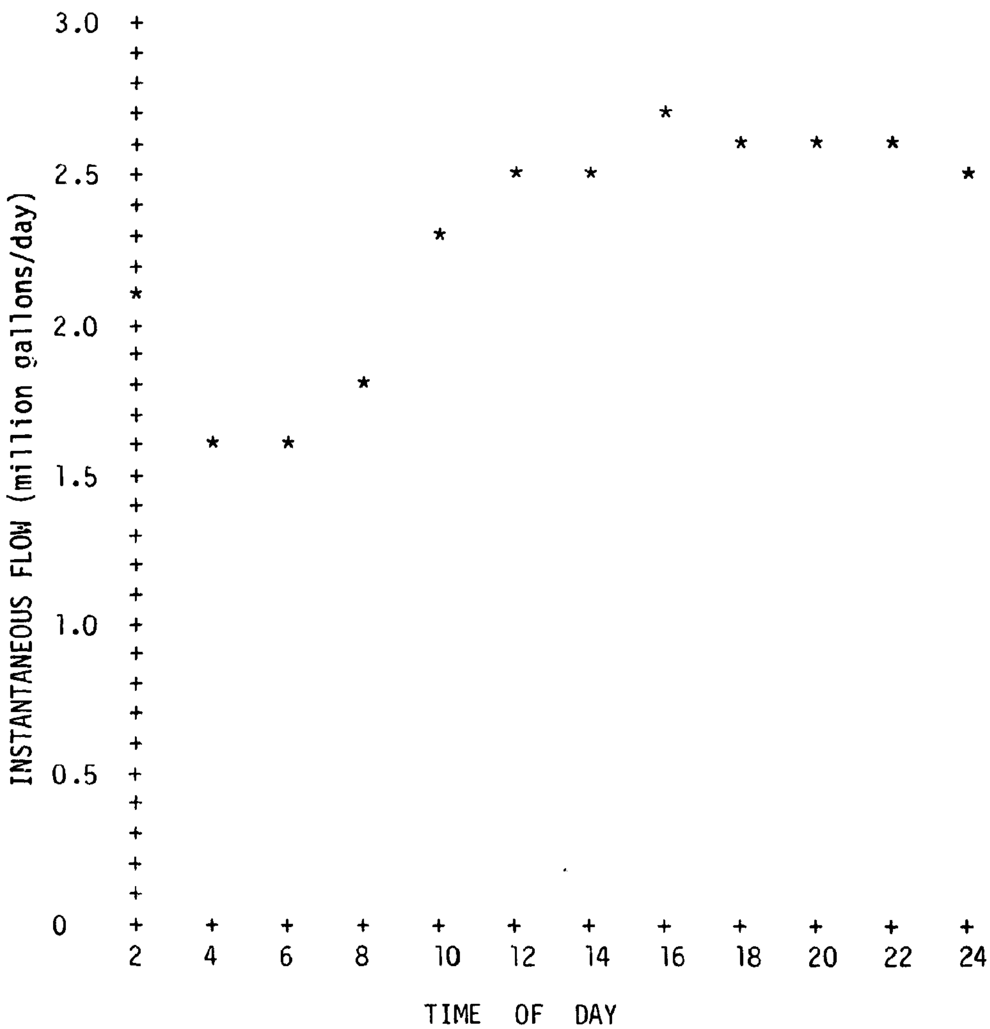


NDSS Treatment Plant
Parshall Flume Data Table

For: July 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	2.1	2.0	1.5	1.7	2.1	2.1	2.1	2.3	2.1	2.1	2.3	2.4	2.1
2	2.4	1.6	1.6	1.45	2.1	2.1	2.1	2.3	2.1	2.8	2.5	2.3	2.1
3	2.1	1.6	1.4	1.25	2.1	2.1	2.1	2.3	1.8	1.8	2.3	2.6	2.0
4	2.3	1.9	1.45	1.4	2.1	2.1	2.1	2.3	2.6	2.6	2.6	2.6	2.2
5	1.75	1.75	1.45	1.6	2.1	2.1	2.1					2.7	
6	2.1	1.75	1.45	1.8	2.9	2.8	2.8					2.6	
7	1.9	1.6	1.75	2.1	2.8	2.6	2.1						
8				1.6	2.6	2.8						2.3	
9	1.8	1.8	1.8	1.8	2.8	2.3	2.3					2.6	
10	1.9	1.6	1.25	1.8	2.8	2.8	2.8	3.0	3.0	3.0	2.6	2.4	2.4
11	1.75	1.45	2.1	2.3	2.8	2.8	2.8	2.9	3.2	3.0	2.8	2.6	2.5
12	1.9	1.6	1.8	2.1	2.6	2.6	2.8	2.6	2.8	2.8	3.0		
13				1.45	1.9	2.1	2.1	2.8	2.8	2.8	2.6	1.6	
14	1.2	1.2	0.9					2.1	2.6	2.8	2.6	2.3	
15	1.8	1.3	1.4	1.9	2.6	2.6	2.8						
16								3.3	4.0	4.0	4.0	3.1	
17	3.1	2.0	1.8					2.8	3.0	3.2	2.1	2.1	
18	1.6	1.6	1.25	1.6	2.6	2.6	2.9	2.9	3.3	3.3	2.9	2.8	2.4
19	2.1	1.25	1.05	2.4	2.25	1.75	2.4	2.4	2.8	3.0	2.8	2.8	2.3
20	2.6	1.75	1.45	1.6	2.1	2.1	2.3	2.6	2.3	2.3	2.1		
21				1.6	1.6	2.3	2.6	2.6	3.1	3.3	3.3		
22				1.6	1.8	2.6	3.1	3.1	3.1	2.3	2.8	2.25	
23	1.75	1.75	2.8	1.6	2.2	3.1	3.3	3.3	3.1	2.8	2.8	2.8	2.6
24	2.25	1.75	1.39	2.1	2.8	3.1	3.1	2.3	1.8	2.1	2.1	2.4	2.3
25	1.75	1.75	2.5	2.8	2.8	3.2	3.1	3.3	2.8	2.3	2.8	2.6	2.6
26	2.6	1.6	1.4	1.6	2.0	2.3	2.3	2.8	2.3	2.6	2.1	2.8	2.2
27	2.6	1.9	1.6	1.25	1.6	1.8	2.3	2.3	1.8	2.1	2.1	2.1	2.0
28	1.6	1.4	1.6	1.8	2.0	2.3	2.3	2.6	2.8	3.0	2.7	2.8	2.2
29	2.6	1.9	1.6					2.8	2.4	1.9	1.75	2.6	
30	1.9	1.4	1.6					2.8	2.45	2.25	1.75	2.6	
31	2.1	1.6	1.4	1.6	2.8	3.2	2.1	2.35	1.9	1.75	2.45	1.9	2.1
AVG.	2.1	1.6	1.6	1.8	2.3	2.5	2.5	2.7	2.6	2.6	2.6	2.5	2.3

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: July 1980

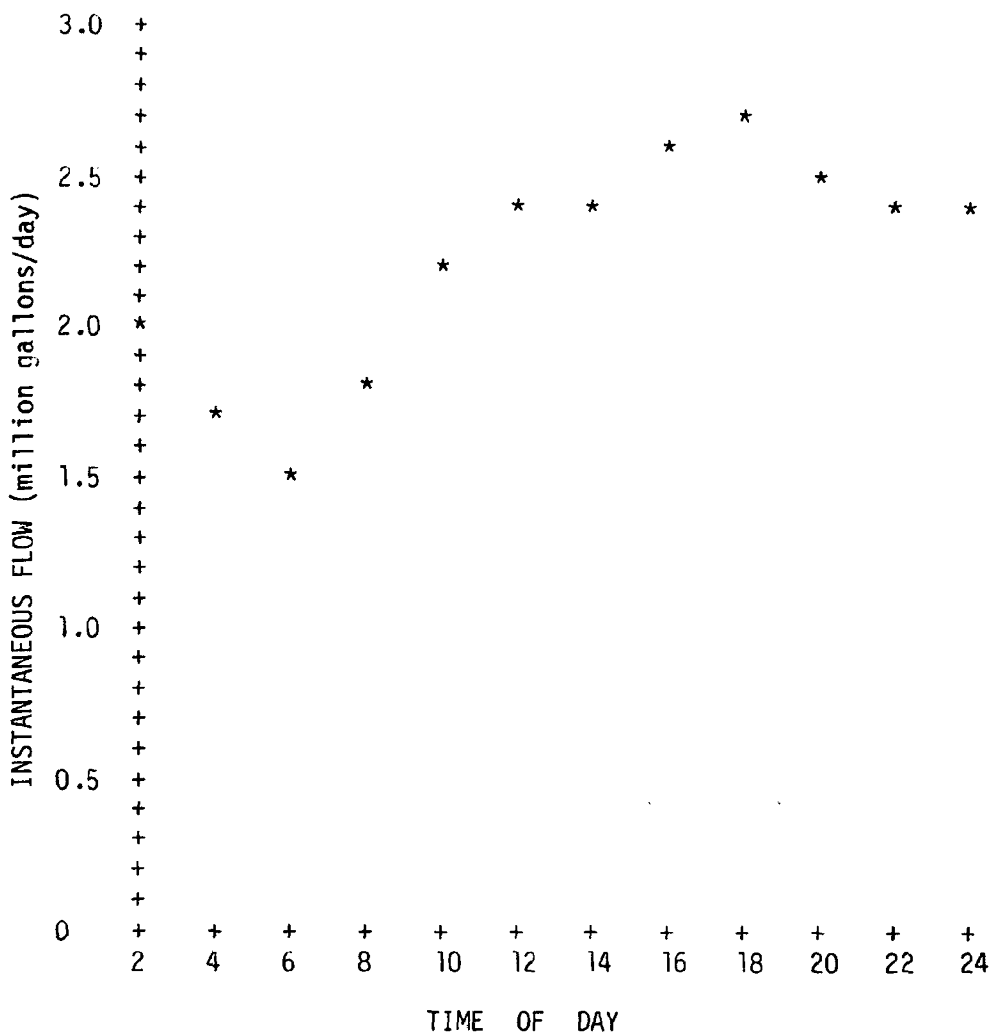


NDSS Treatment Plant
Parshall Flume Data Table

For: August 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	1.6	1.45	1.25	1.8	1.8	1.8	2.3	2.9	2.7	2.45	2.25	1.9	2.0
2	1.6	1.25	1.25	1.8	2.8	3.0	2.8	2.88	2.8	2.5	2.35	2.18	2.3
3	1.45	1.6	1.4	2.3	1.8	2.0	2.8	3.0	3.7	3.3	3.3	2.3	2.4
4	1.8	1.45	1.25	1.8	2.8	2.8	2.3	2.6	3.3	2.8	2.1	2.1	2.3
5	1.6	1.6	1.25	2.0	2.0	2.0	1.6	1.6	2.3	3.1	3.1	2.1	2.0
6	1.8	1.6	1.25	1.6	1.6	1.8	1.6	2.7	2.9	2.6	2.1	2.8	2.0
7	2.1	1.4	1.2	1.2	2.0	3.0	3.2	2.1	2.8	2.7	2.1	2.3	2.2
8	1.8	1.4	1.6	1.4	1.8	2.0	2.1	2.6	2.8	2.3	2.6	2.6	2.1
9	2.3	2.1	1.8	1.8	1.8	2.0	2.1	2.1	2.3	2.3	2.3	2.3	2.1
10	2.3	1.6	1.6	1.4	1.4	1.8	2.1	2.8	2.8	2.1	2.1	3.1	2.1
11	2.1	1.6	1.4	2.0	2.3	3.6	2.6	2.6	2.3	2.3	2.3	2.3	2.3
12	2.3	1.8	1.88	1.8	2.3	2.8	2.1	1.8	2.3	2.3	2.3	2.5	2.2
13	1.75	1.75	2.45	1.4	1.8	2.8	2.6	2.1	2.3	2.3	2.3	2.0	2.1
14	1.75	1.45	1.19	1.6	2.0	2.1	2.1	2.1	2.1	1.8	1.8	2.7	1.9
15	1.9	1.25	1.0	1.6	2.3	2.8	2.6	2.6	2.6	2.8	2.1	1.75	2.1
16	1.45	1.25	0.9	1.8	2.3	2.8	2.8	3.0	2.8	2.6	2.6	2.4	2.2
17	1.75	1.25	1.1	1.6	2.3	2.3	2.8	2.9	2.9	2.8	2.6	2.7	2.3
18	1.75	1.19	1.0	1.6	1.8	2.3	2.3	2.8	2.9	2.9	2.9	2.1	2.1
19	2.0	1.8	1.8	1.6	2.2	2.3	2.3	2.9	2.9	2.9	2.9	2.8	2.4
20	2.3	1.8	1.6	1.6	2.0	2.3	2.8	2.8	2.8	2.8	2.8	2.8	2.4
21	2.8	2.1	2.1	1.6	1.8	2.3	2.3	2.8	2.8	2.4	2.7	2.6	2.4
22	2.8	2.8	2.1	1.6	1.8	2.3	2.3	3.2	3.0	2.7	2.25	3.0	2.5
23	2.8	2.8	2.6	2.6	3.0	3.0	3.0	3.3	3.1	3.1	2.3	2.8	2.9
24	2.1	1.8	1.3	1.4	2.3	2.8	2.8	2.8	2.1	1.75	1.75	1.9	2.1
25	1.9	1.45	1.25	1.4	2.8	2.8	2.8	2.6	2.4	2.6	2.45	2.4	2.2
26	1.9	1.45	1.25	1.6	2.8	2.8	2.8	2.7	2.6	1.75	1.75	1.9	2.1
27	1.9	1.45	1.25	1.6	2.6						3.1	2.8	
28	2.1	1.6	1.25	2.0	2.3	2.8	2.8	2.8	2.9	2.1	2.1	2.1	2.2
29	2.1	1.6	1.25	2.7	2.8	1.8	2.6	2.6	2.9	2.8	2.1	2.1	2.3
30	1.9	1.6	1.25	2.3	2.7	2.1	2.1	2.1	2.3	2.6	2.6	2.8	2.2
31	2.8	2.6	1.9	2.8	2.3	1.75	1.75	2.6	2.8	2.3	2.1	2.8	2.4
AVG.	2.0	1.7	1.5	1.8	2.2	2.4	2.4	2.6	2.7	2.5	2.4	2.4	2.2

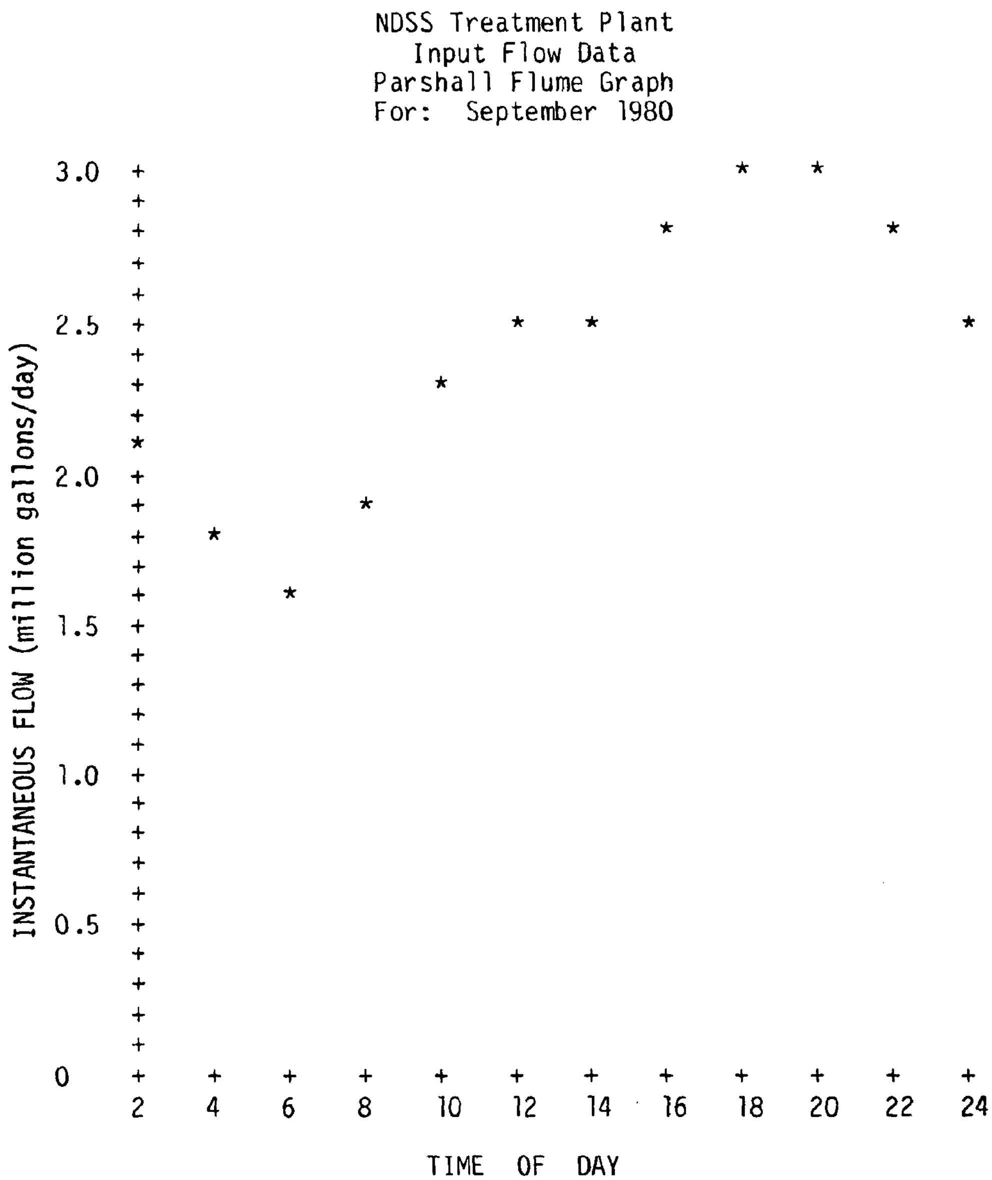
NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: August 1980



NDSS Treatment Plant
Parshall Flume Data Table

For: September 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	2.6	1.8	1.4	1.9	1.6	1.6	1.8	2.8	3.2	2.8	2.1	2.8	2.2
2	2.8	2.1	1.6	2.8	2.8	2.1	2.1	2.1	2.8	2.9	3.0	2.6	2.5
3	2.6	2.1	1.6	2.5	1.8	2.6	2.6	2.3	2.8	3.1	2.8	2.9	2.5
4	2.9	2.6	2.4	1.6	2.1	2.8	2.1	2.1	2.8	2.8	3.0	2.8	2.5
5	2.1	2.1	1.6	2.0	2.1	2.1	2.8	2.8	2.8	2.8	3.3	1.65	2.3
6	1.25	1.1	1.2	2.8	2.8	2.9	3.0	4.7	4.5	3.3	2.6	3.6	2.8
7	2.6	2.6	2.6	1.05	1.4	1.7	2.3	3.1	3.1	2.3	2.3	1.9	2.2
8	1.45	1.45	1.25	2.0	2.3	2.6	2.1	3.0	3.2	3.0	2.8	2.3	2.3
9	2.8	2.8	2.8	3.0	3.0	3.0	2.9	3.0	3.4	3.4	3.4	2.3	3.0
10	1.6	1.4	1.25	1.6	1.8	2.1	2.1	3.0	3.0	2.8	2.6	2.3	2.1
11	1.5	1.25	1.39	1.25	2.0	2.3	2.3	2.8	2.8	2.8	2.3	2.6	2.1
12	2.1	1.8	1.4	1.8	1.8	2.1	2.1	2.8	2.8	3.2	3.2	2.8	2.3
13	2.1	1.8	1.4	1.4	1.8	2.1	2.1	2.8	3.2	3.2	2.8	2.3	2.3
14	1.8	1.8	1.6	1.6	1.8	2.1	2.6	3.0	3.0	2.6	2.6	2.4	2.2
15	1.45	1.25	1.2	1.6	1.6	1.8	2.0	2.3	2.8	2.8	2.8	2.1	2.0
16	2.1	1.6	1.4	1.6	2.3	2.8	2.8	2.1	2.8	2.6	2.4	2.6	2.3
17	1.8	1.4	1.2	1.6	2.8	2.8	2.8	2.6	2.6	2.6	2.6	2.6	2.3
18	2.1	1.4	1.05	1.6	3.6	3.6	3.6	2.8	3.6	3.6	3.6	2.6	2.8
19	2.1	1.6	0.9	2.1	2.8	2.9	2.9	2.6	2.8	2.8	2.8	2.8	2.4
20	2.1	1.6	1.25	2.3	2.8	3.0	3.0	3.0	3.3	2.9	2.3	2.1	2.5
21	1.9	1.6	1.4	1.6	2.1	2.8	2.9	2.8	2.8	2.6	2.1	2.1	2.2
22	2.1	1.6	1.4	2.3	2.8	2.6	2.6	2.3	3.1	2.6	2.0	2.0	2.3
23	1.4	1.4	1.45	2.3	2.3	2.6	2.6	2.6	2.6	2.9	2.8	2.8	2.3
24	2.1	2.1	2.8					1.6	2.3	2.8	1.8	2.1	
25	2.1	1.6	1.4	2.1	2.6	2.6	2.6	2.8	2.7	2.6	2.0	2.6	2.3
26	2.0	1.8	1.7	1.8	2.1	2.1	2.1	2.8	3.1	3.3	3.3	3.3	2.5
27	3.3	3.1	2.0	2.0	2.3	2.8	3.0	3.3	3.1	3.0	2.8	2.8	2.8
28	2.8	2.6	1.6	1.4	1.8	2.3	2.5	3.1	3.6	3.9	3.2	2.9	2.6
29	2.7	1.9	1.6	1.8	2.5	3.1							
30								3.2	3.5	4.5	4.5	3.0	
31													
AVG.	2.1	1.8	1.6	1.9	2.3	2.5	2.5	2.8	3.0	3.0	2.8	2.5	2.4

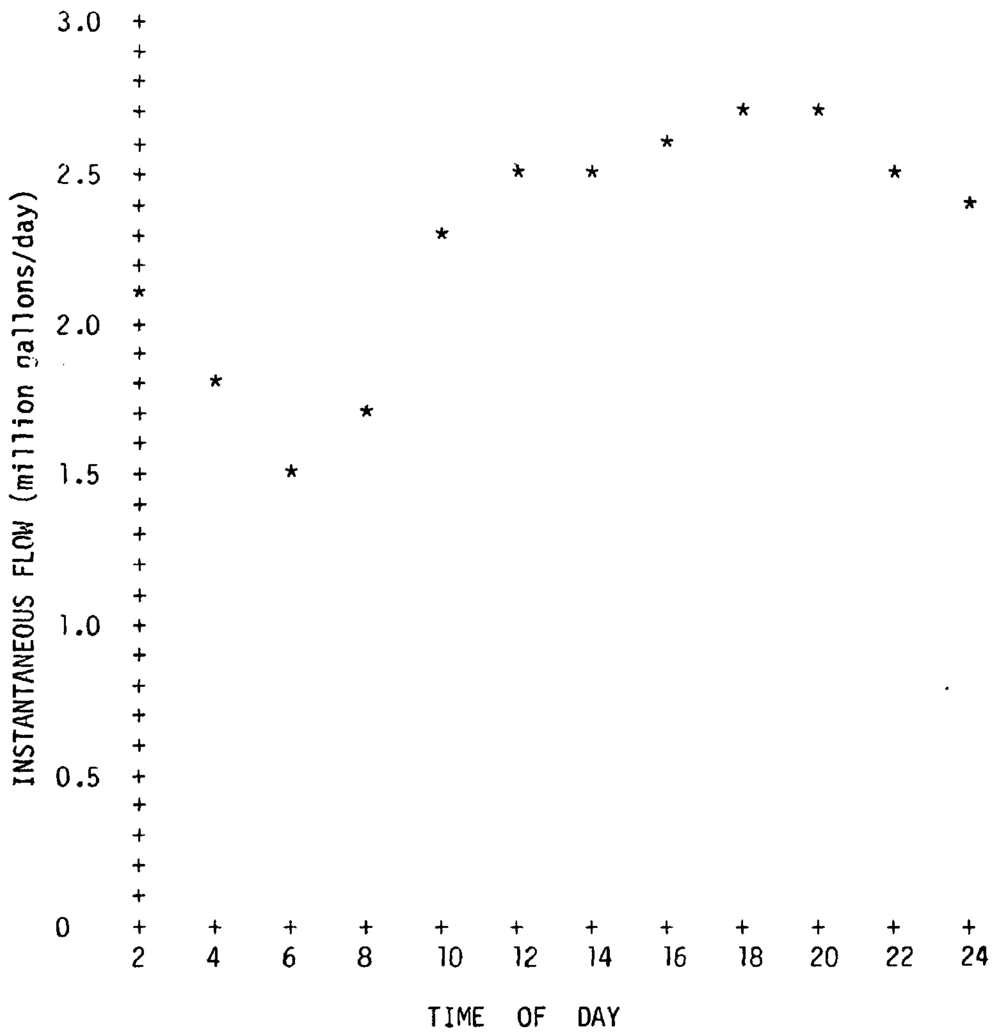

NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: September 1980
INSTANTANEOUS FLOW (million gallons/day)
3.0
2.5
2.0
1.5
1.0
0.5
0
2
4
6
8
10
12
14
16
18
20
22
24
TIME OF DAY

NDSS Treatment Plant
Parshall Flume Data Table

For: October 1980

DAY	2	4	6	8	10	12	14	16	18	20	22	24	AVG.
1	2.6	2.6	2.6	3.0	3.0	2.7	2.6	2.1	2.8	2.8	2.8	2.6	2.7
2	1.9	1.9	1.75	2.1	2.5	2.8	2.6	2.6	2.6	2.9	2.9	2.8	2.4
3	2.3	2.25	2.8	1.8	2.3	2.6	2.7	2.5	2.5	2.8	2.8	2.8	2.5
4	2.8	2.1	1.8	1.5	1.8	2.1	2.1	2.6	2.6	2.6	2.6	2.8	2.3
5	1.75	1.65	1.25	1.5	1.8	2.1	2.3	2.8	2.8	2.8	2.3	2.6	2.1
6	2.0	1.6	1.4	1.6	2.7	2.9	2.9	2.9	2.9	2.9	3.0	3.0	2.5
7	2.6	2.6	2.1	1.4	2.0	2.1	2.3	2.8	2.9	2.1	2.1	2.6	2.3
8	2.0	1.6	1.4	1.4	1.8	2.1	2.1	1.9	2.8	2.8	2.6	2.6	2.1
9	2.1	1.6	1.4	1.4	2.0	2.8	2.6	2.1	2.6	2.6	2.45	2.6	2.2
10	2.4	2.1	1.8	1.6	2.8	2.8	2.8	2.7	2.1	1.75	2.25	2.5	2.3
11	1.9	1.45	1.25	1.6	2.8	2.8	3.0	2.6	2.8	2.3	2.3	2.3	2.3
12	2.1	1.6	1.6	1.6	1.9	2.1	2.8	2.8	2.8	2.8	2.6	1.9	2.2
13	1.9	1.25	1.05	1.6	3.0	3.0	2.8	2.6	2.6	2.6	2.6	2.1	2.3
14	1.6	1.25	1.05	1.4	2.0	2.1	2.4	2.6	2.6	2.0	2.0	2.0	1.9
15	1.6	1.05	0.9	1.6	2.8	2.8	3.0	3.0	2.9	2.8	2.1	2.1	2.2
16	1.8	1.25	0.9	2.1	2.1	2.6	2.6	2.8	3.1	2.8	2.1	2.1	2.2
17	1.8	1.25	1.05	2.6	1.9	1.9	1.8	2.1	2.6	2.8	2.1	2.7	2.1
18	2.6	1.8	1.35	1.4	1.4	1.8	2.1	2.9	3.0	2.6	2.1	2.1	2.1
19	2.1	2.1	1.4	1.45	1.75	1.75	1.75	2.1	2.6	2.3	2.1	2.6	2.0
20	2.6	2.3	1.25	1.45	3.1	2.9	1.9	2.3	2.3	2.2	2.2	2.2	2.2
21	2.2	2.2	1.5	1.25	2.45	2.4	2.6	2.6	2.8	3.3	2.4	2.8	2.4
22	2.8	1.9	1.25	1.6	2.9	2.3	2.1	2.6	2.8	3.3	3.8	2.8	2.5
23	2.1	2.1	1.4	1.6	2.1	3.0	2.8	2.8	2.9	3.3	3.2	2.25	2.5
24	1.45	1.25	1.19	1.8	2.9	3.0	2.6					2.1	
25	2.1	1.8	1.6	1.8	2.3	3.0	2.8	2.8	2.8	2.1	2.1	1.45	2.2
26	1.25	1.25	0.9	2.0	2.3	2.8	2.6	2.8	3.0	2.8	2.8	2.8	2.3
27	2.1	1.25	1.45	2.1	2.6	2.9	2.9	2.8	2.9	2.8	2.8	1.75	2.4
28	1.75	1.25	1.1	1.05	2.0	2.1	2.3	2.3	2.3	2.8	2.6	2.1	2.0
29	1.25	1.1	1.2	1.05	1.9	2.1	2.3	2.8	2.8	2.8	2.9	2.6	2.1
30	2.6	1.9	1.6	1.8	1.8	2.3	2.3	2.8	2.8	2.8	2.8	2.3	2.3
31	1.8	3.0	2.9										
AVG.	2.1	1.8	1.5	1.7	2.3	2.5	2.5	2.6	2.7	2.7	2.5	2.4	2.3

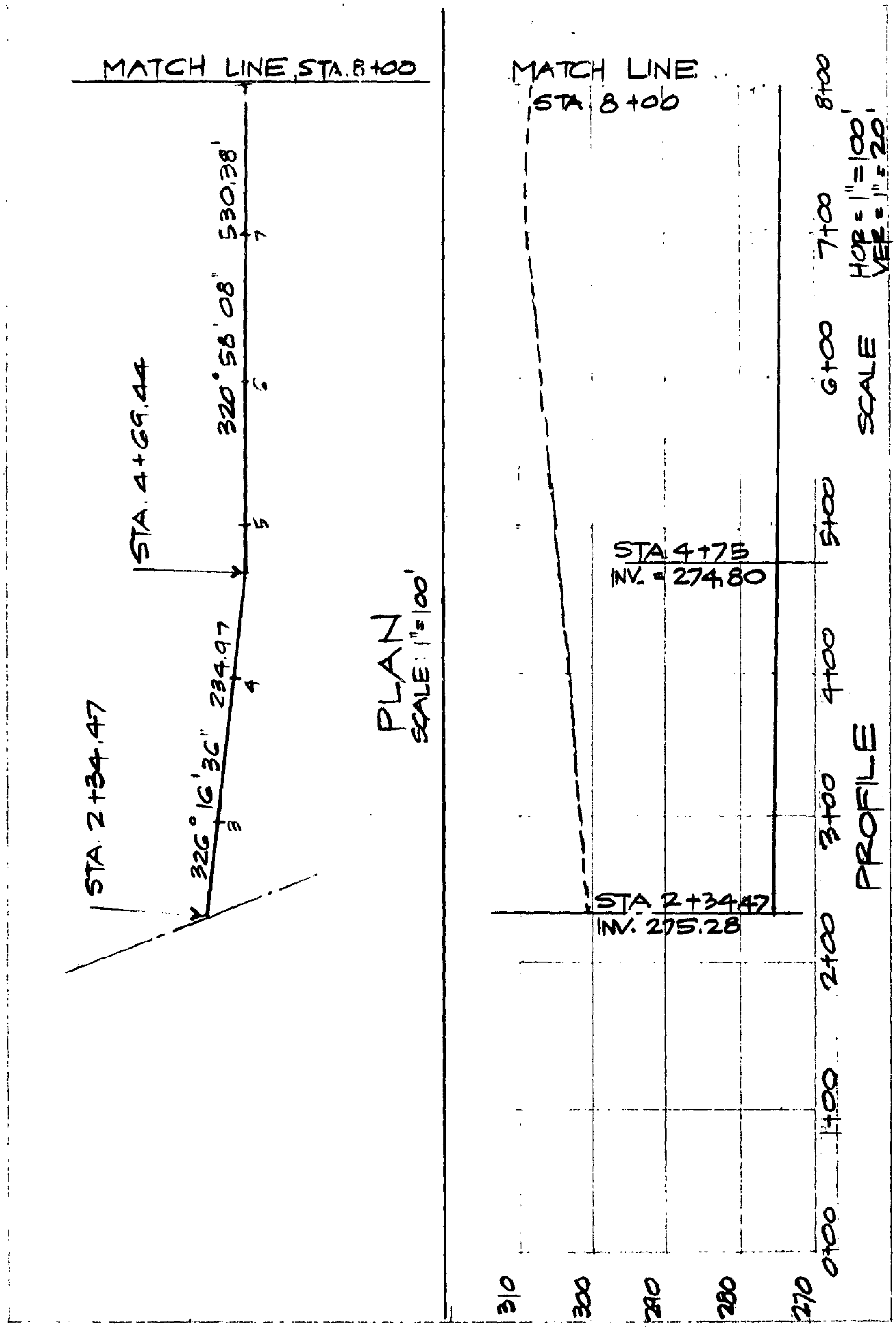
NDSS Treatment Plant
Input Flow Data
Parshall Flume Graph
For: Octboer 1980



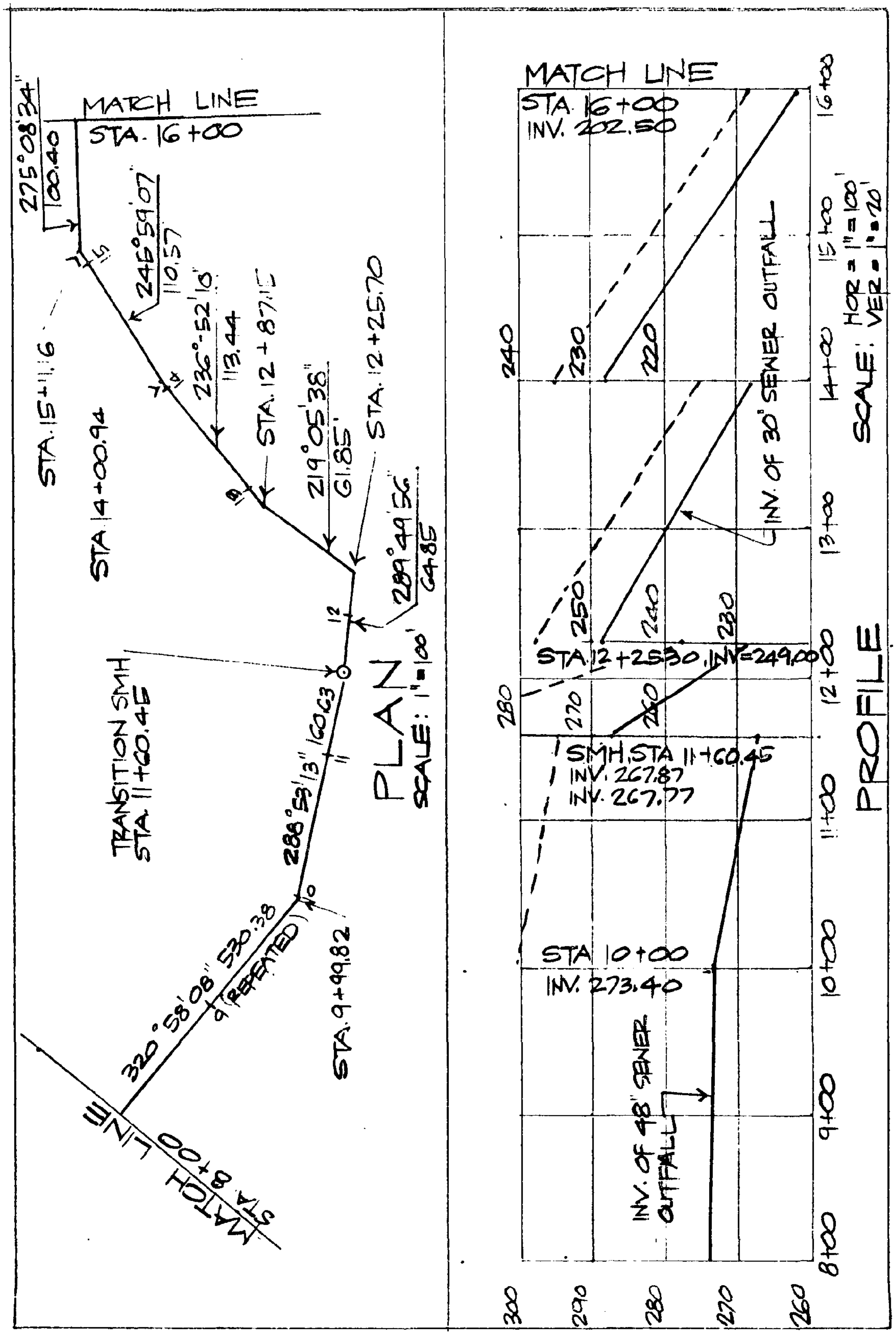
APPENDIX C

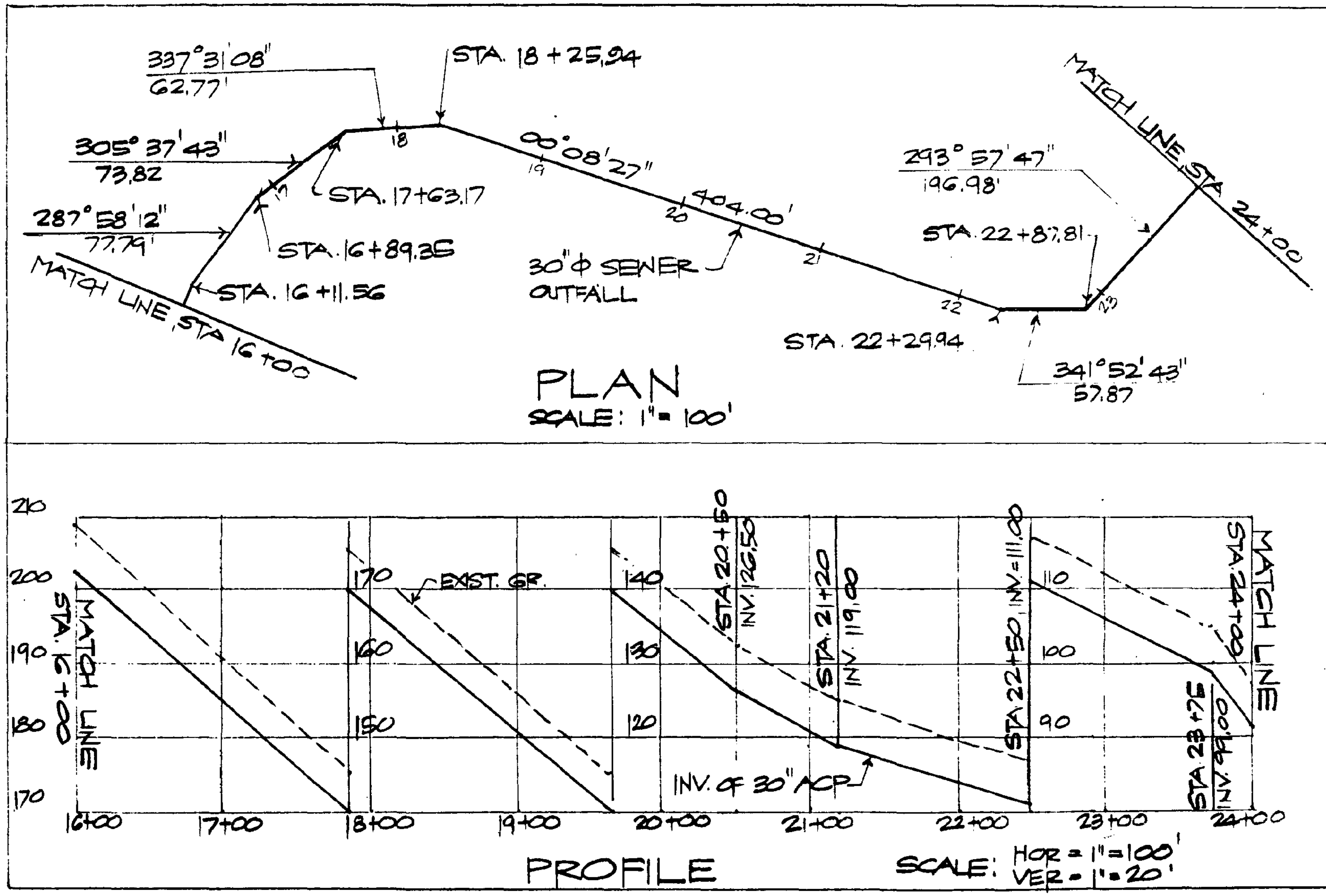
OUTFALL LINE PLAN AND PROFILE

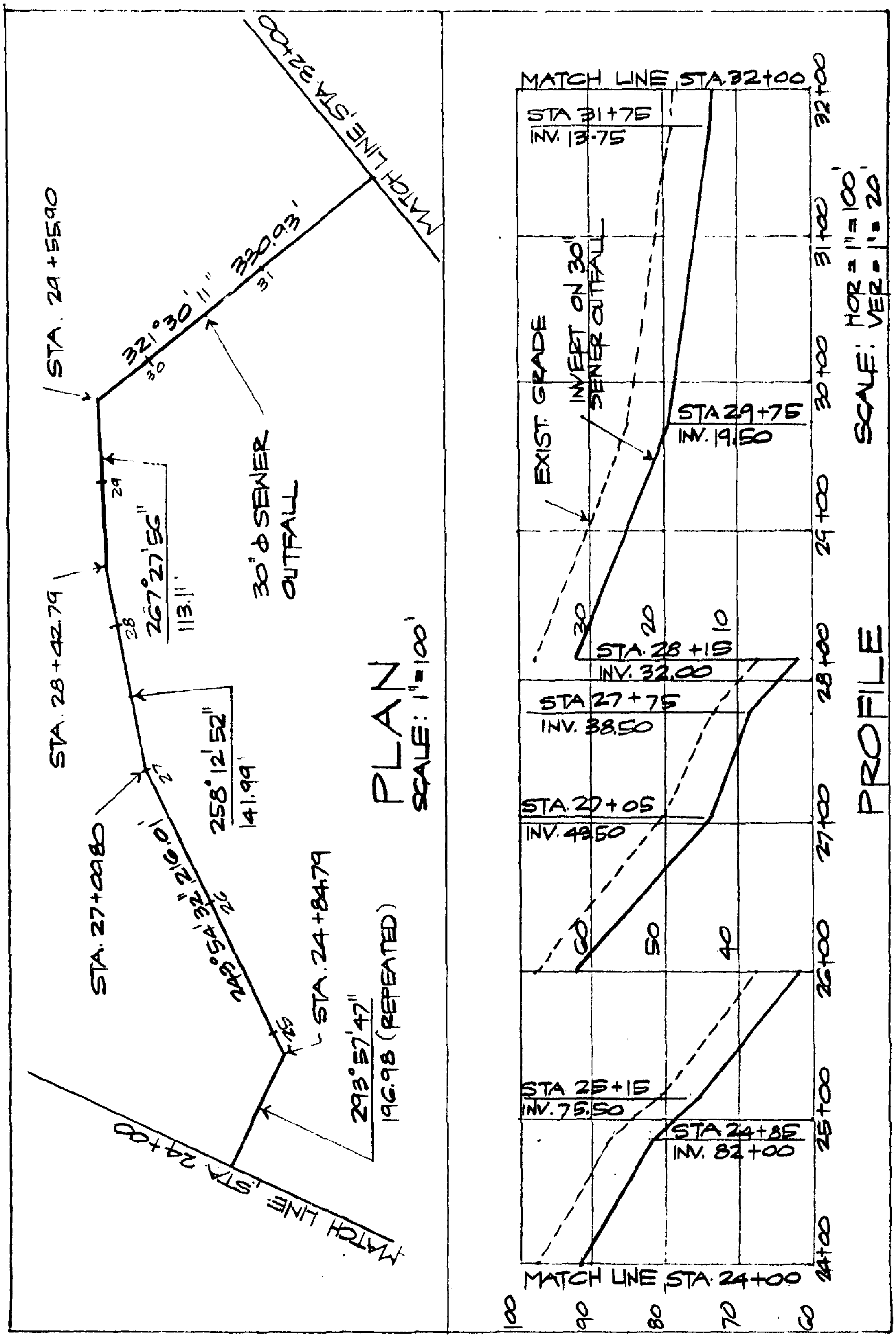
Sketches of the outfall line Plan and Profile are reproduced from As Built Drawings, Northern District Sewarage System, Austin, Smith and Associates, Incorporated, NAVFAC drawings 73-04-819 ff, dtd. 7/23/75.



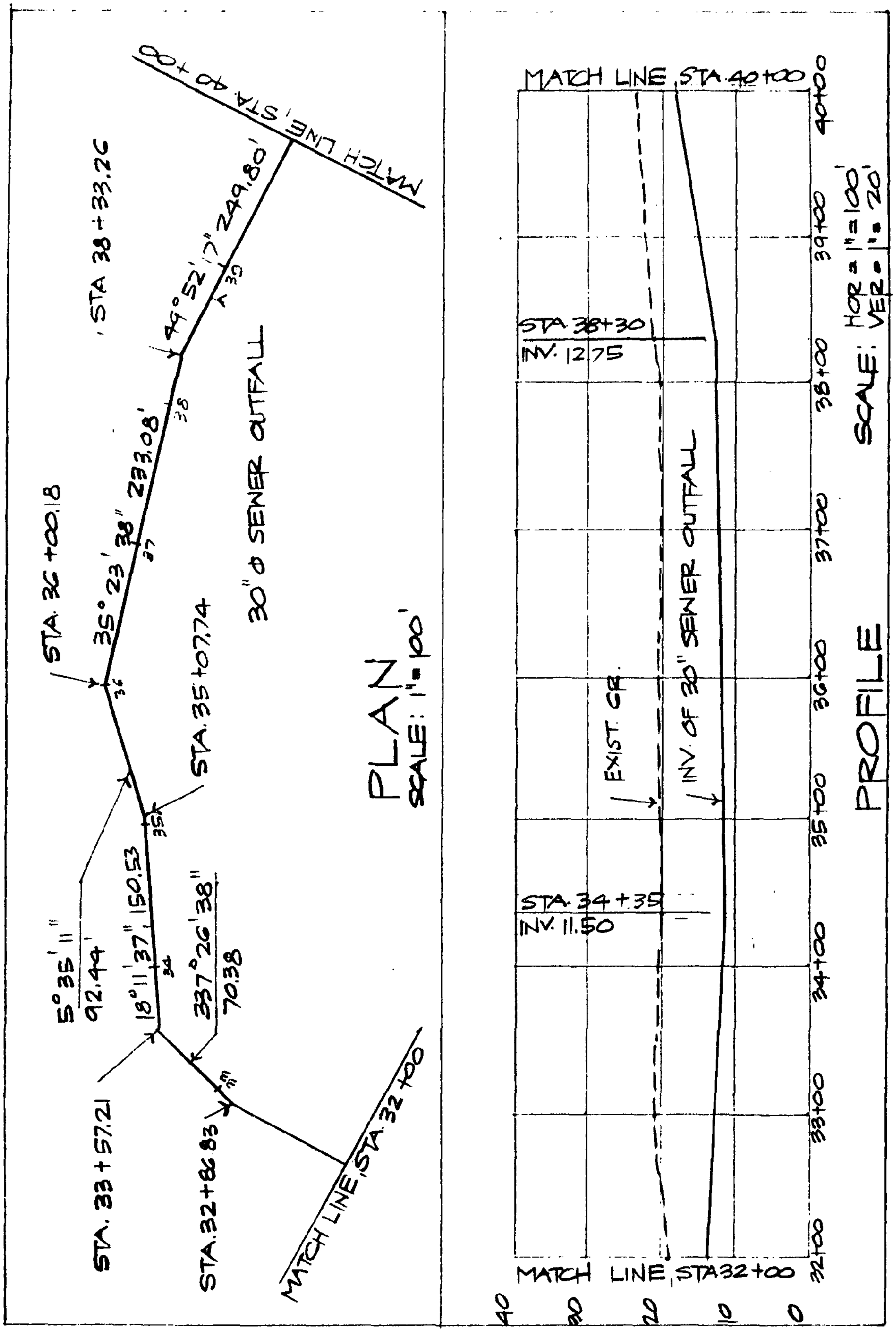
MATCH LINE STA. 8+00
530.38'
320°58'08"
STA. 4+69.44
326°16'36"
234.97
STA. 2+34.47
PLAN
SCALE: 1"=100'
MATCH LINE
STA. 8+00
STA 4+75
INV. = 274.80
STA 2+34.47
INV. 275.28
0+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
310
300
290
280
270
PROFILE
SCALE
HOR = 1"=100'
VER = 1"=20'

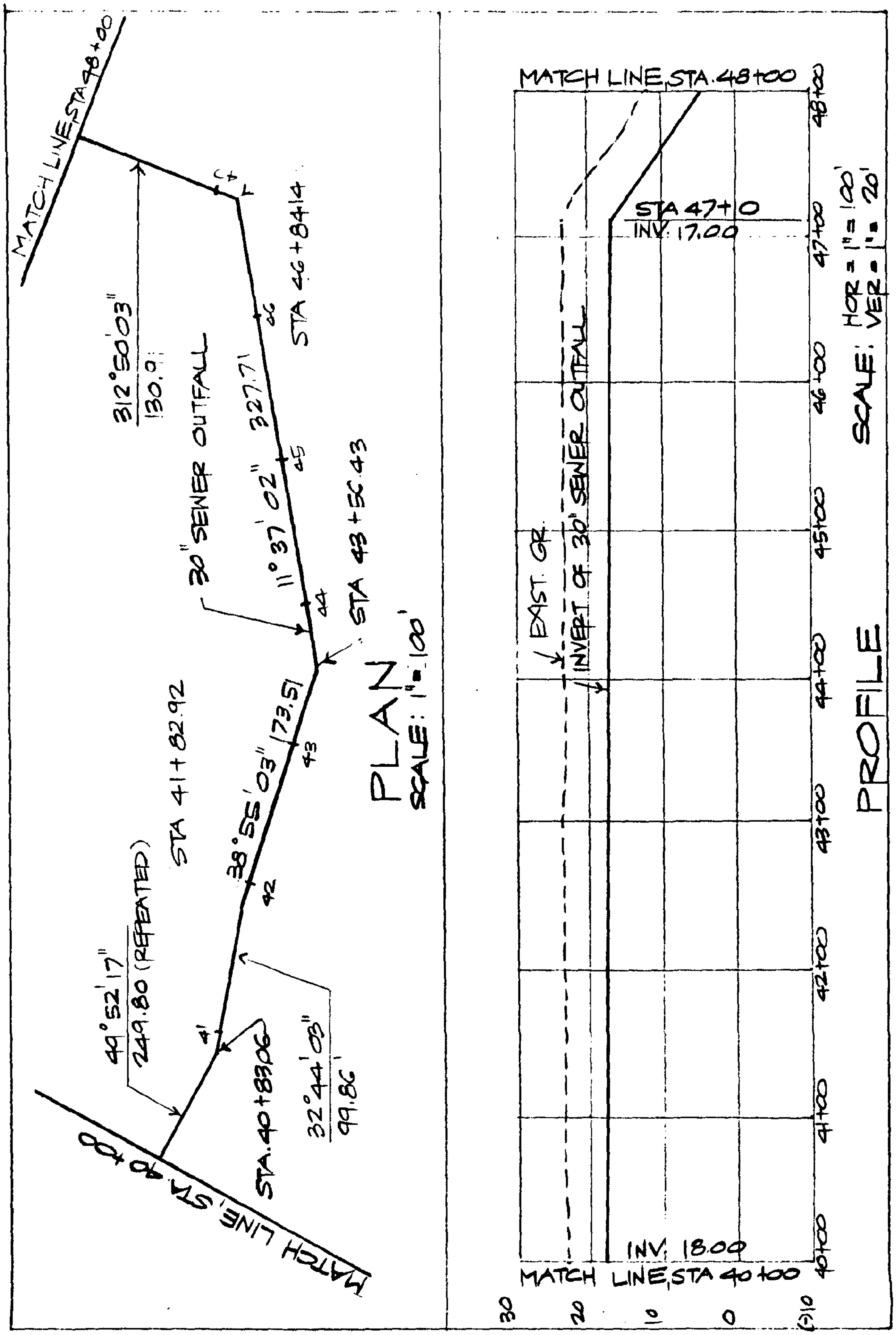

MATCH LINE
STA. 16+00
275°08'24"
100.40
245°59'07"
110.57
236°52'10"
113.44
STA. 15+11.6
STA. 14+00.94
STA. 12+87.15
219°05'38"
61.85'
STA. 12+25.70
289°49'56"
64.85
TRANSITION SMH
STA. 11+60.45
288°53'13" 160.63
PLAN
SCALE: 1"=100'
320°58'08" 530.38
(REPEATED)
STA. 9+99.82
MATCH LINE
STA. 8+00
MATCH LINE
STA. 16+00
INV. 202.50
INV. OF 30" SEWER OUTFALL
SCALE: HOR = 1"=100' VER = 1"=20'
STA. 12+25.30, INV=249.00
SMH STA 11+60.45
INV. 267.87
INV. 267.77
STA 10+00
INV. 273.40
INV. OF 48" SEWER OUTFALL
PROFILE
8+00
9+00
10+00
11+00
12+00
13+00
14+00
15+00
16+00
300
290
280
270
260
250
240
230
220


337°31'08"
62.77'
STA. 18 + 25.94
305° 37' 43"
73.82
STA. 17+63.17
287° 58' 12"
77.79'
STA. 16+89.35
STA. 16 +11.56
MATCH LINE STA 16+00
00°08'27"
404.00'
30"Φ SEWER OUTFALL
293° 57' 47"
196.98'
STA. 22+87.81
MATCH LINE STA 24+00
STA. 22+29.94
341°52' 43"
57.87
PLAN
SCALE: 1"= 100'
MATCH LINE STA. 16+00
EXIST. GR.
STA. 20+50 INV. 126.50
STA. 21+20 INV. 119.00
STA. 22+50, INV.=111.00
STA. 23+75 INV. 99.00
MATCH LINE STA. 24+00
INV. OF 30" ACP
16+00
17+00
18+00
19+00
20+00
21+00
22+00
23+00
24+00
PROFILE
SCALE: HOR = 1"=100' VER = 1"=20'

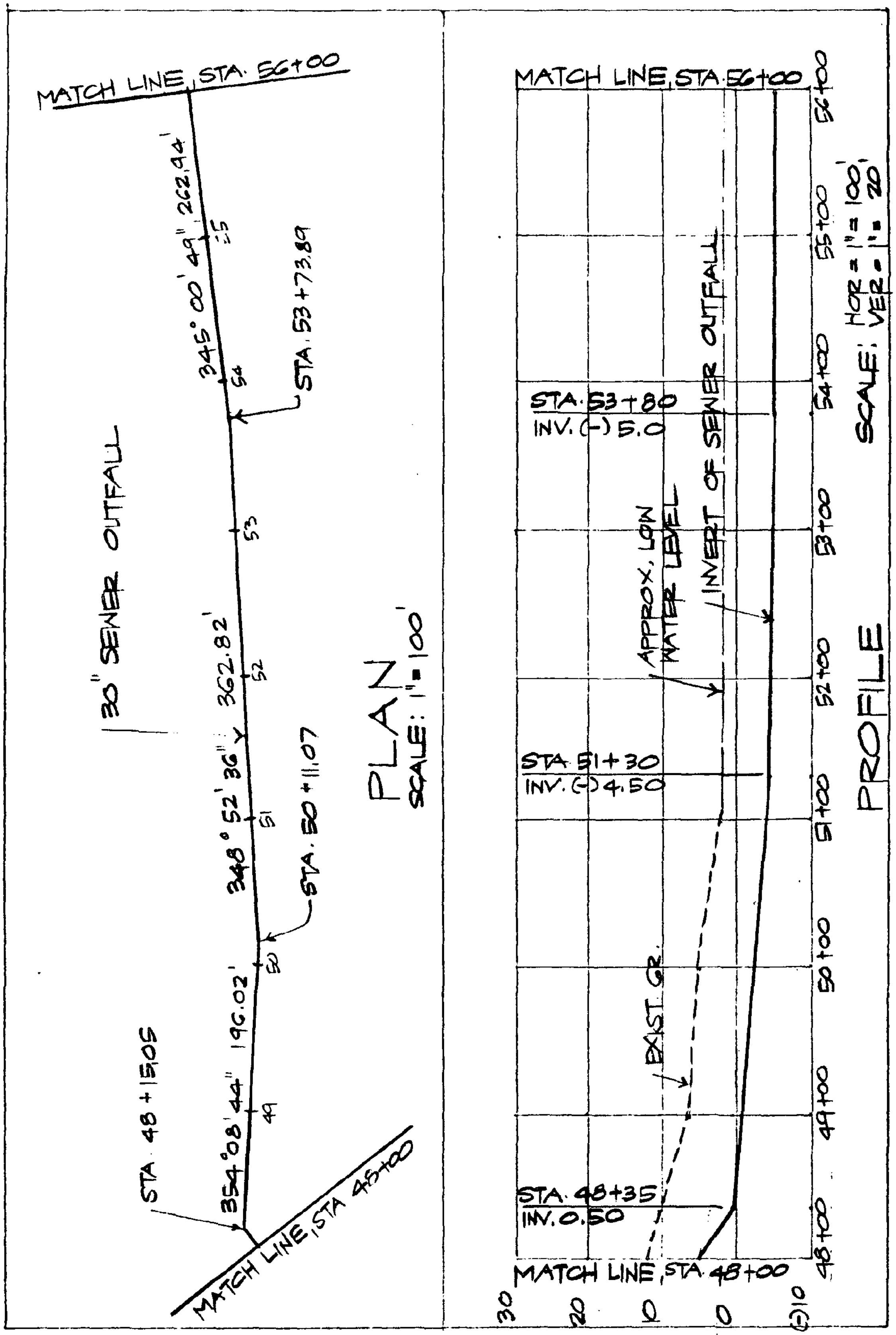


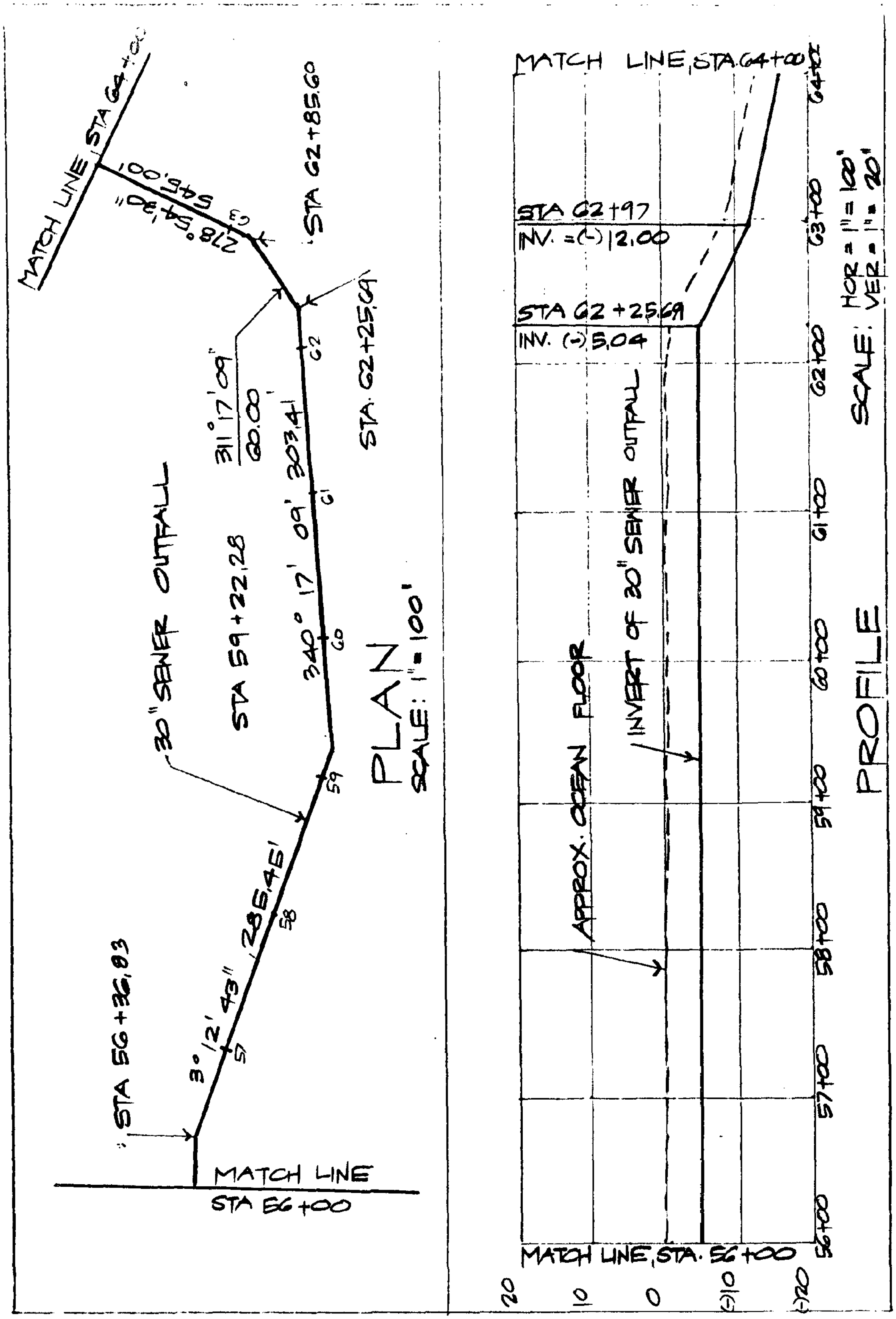
PLAN
SCALE: 1"=100'
MATCH LINE STA 32+00
MATCH LINE STA 24+00
STA. 29+55.90
321°30'11" 330.93'
30" ϕ SEWER OUTFALL
STA. 28+42.79
267°27'56"
113.11'
258°12'52"
141.99'
STA. 27+00.80
249°54'32" 216.01'
STA. 24+84.79
293°57'47"
196.98 (REPEATED)
PROFILE
SCALE: HOR = 1"=100'
VER = 1"= 20'
STA 31+75
INV. 13.75
EXIST. GRADE
INVERT ON 30" SEWER OUTFALL
STA 29+75
INV. 19.50
STA. 28+15
INV. 32.00
STA 27+75
INV. 38.50
STA. 27+05
INV. 48.50
STA. 25+15
INV. 75.50
STA. 24+85
INV. 82+00

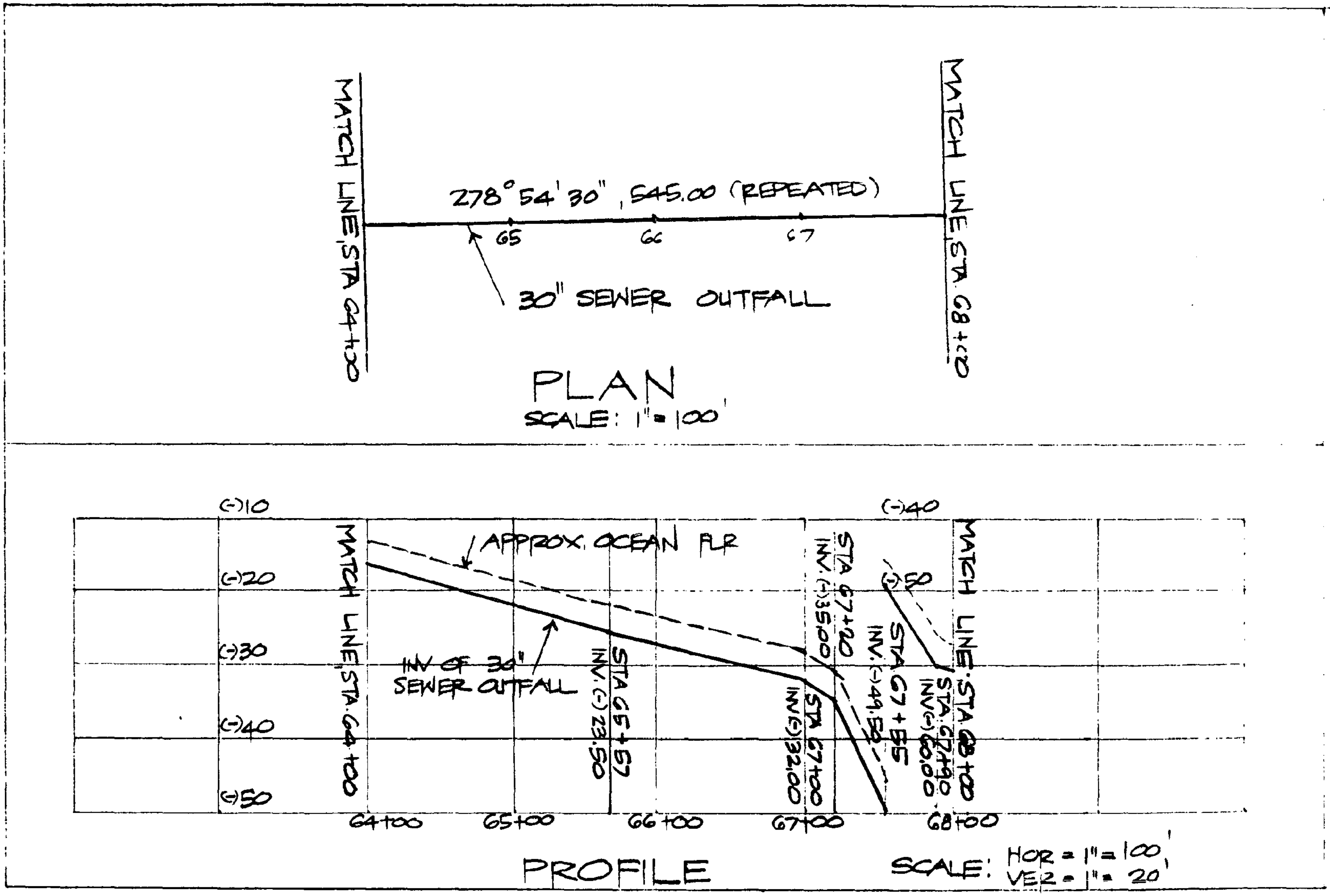

MATCH LINE, STA. 40+00
STA 38+33.26
49°52'17" 249.80'
STA. 36+00.18
35°23'38" 233.08'
30"ø SEWER OUTFALL
STA. 35+07.74
5°35'11"
92.44'
18°11'37" 150.53
337°26'38"
70.38
STA. 33+57.21
STA.32+86.83
MATCH LINE, STA. 32+00
PLAN
SCALE: 1"=100'
MATCH LINE, STA. 40+00
STA. 38+30
INV. 12.75
STA. 34+35
INV. 11.50
EXIST. GR.
INV. OF 30" SEWER OUTFALL
MATCH LINE, STA32+00
40+00
39+00
38+00
37+00
36+00
35+00
34+00
33+00
32+00
40
30
20
10
0
PROFILE
SCALE: HOR = 1"=100'
VER = 1"=20'



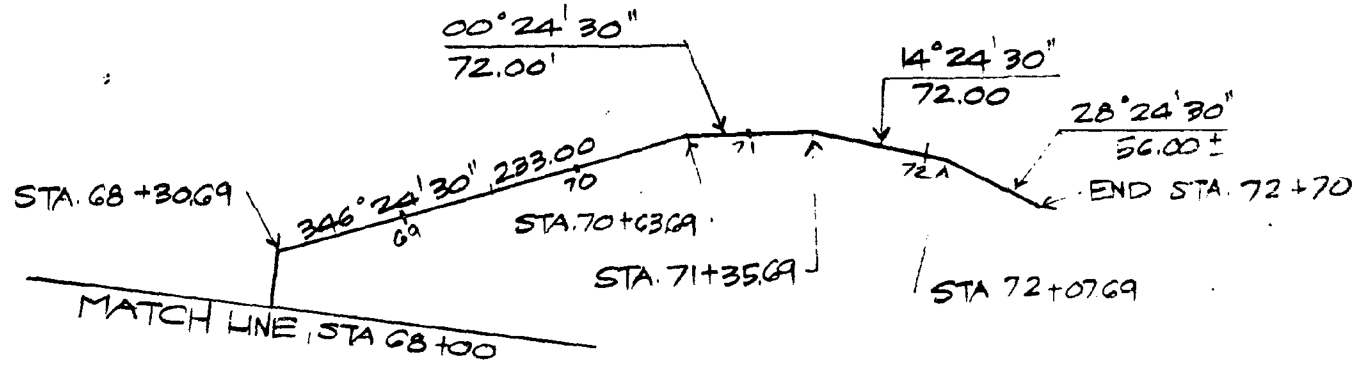
PLAN
SCALE: 1" = 100'
MATCH LINE, STA 48+00
MATCH LINE, STA 40+00
312°50'03"
130.9'
STA 46+84.14
30" SEWER OUTFALL
11°37'02"
327.71
STA 43+56.43
38°55'03"
173.51
STA 41+82.92
49°52'17"
249.80 (REPEATED)
STA. 40+83.06
32°44'03"
99.86'
41
42
43
44
45
46
PROFILE
SCALE: HOR = 1" = 100'
VER = 1" = 20'
MATCH LINE, STA. 48+00
MATCH LINE, STA 40+00
STA 47+0
INV. 17.00
INV. 18.00
EXIST. GR.
INVERT OF 30" SEWER OUTFALL
40+00
41+00
42+00
43+00
44+00
45+00
46+00
47+00
48+00
30
20
10
0
(-)10


MATCH LINE, STA. 56+00
345° 00' 49" 262.94'
55
54
STA. 53+73.89
30" SEWER OUTFALL
53
362.82'
52
348° 52' 36"
51
STA. 50+11.07
50
196.02'
354° 08' 44"
49
STA. 48+15.05
MATCH LINE, STA 48+00
PLAN
SCALE: 1" = 100'
MATCH LINE, STA. 56+00
STA. 53+80
INV. (-) 5.0
STA. 51+30
INV. (-) 4.50
STA. 48+35
INV. 0.50
APPROX. LOW WATER LEVEL
INVERT OF SEWER OUTFALL
EXIST. GR.
MATCH LINE, STA. 48+00
56+00
55+00
54+00
53+00
52+00
51+00
50+00
49+00
48+00
30
20
10
0
(-)10
PROFILE
SCALE: HOR = 1" = 100'
VER = 1" = 20'

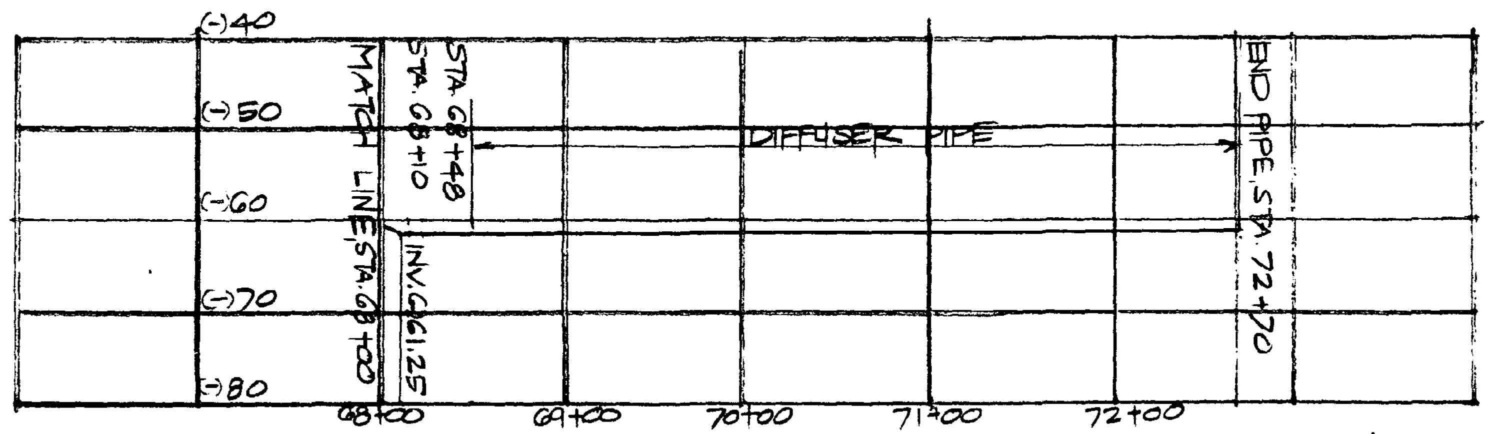

MATCH LINE STA 64+00
S46°00'
278°54'30"
STA 62+85.60
STA 62+25.69
31°17'09"
60.00'
303.41'
340° 17' 09'
STA 59+22.28
30" SEWER OUTFALL
PLAN
SCALE: 1" = 100'
STA 56+36.83
3° 12' 43"
286.45'
MATCH LINE
STA 56+00
MATCH LINE, STA 64+00
STA 62+97
INV. = (-) 12.00
STA 62+25.69
INV. (-) 5.04
INVERT OF 30" SEWER OUTFALL
APPROX. OCEAN FLOOR
MATCH LINE, STA. 56+00
SCALE: HOR = 1" = 100'
VER = 1" = 20'
PROFILE
20
10
0
(-)10
(-)20
56+00
57+00
58+00
59+00
60+00
61+00
62+00
63+00
64+00



MATCH LINE, STA 64+00
278° 54' 30", 545.00 (REPEATED)
65
66
67
30" SEWER OUTFALL
MATCH LINE, STA. 68+00
PLAN
SCALE: 1" = 100'
(-)10
(-)20
(-)30
(-)40
(-)50
MATCH LINE, STA. 64+00
APPROX. OCEAN FLR
INV OF 30" SEWER OUTFALL
STA 65+57
INV. (-) 23.50
STA 67+00
INV (-) 32.00
STA 67+20
INV. (-) 35.00
STA 67+55
INV. (-) 49.50
(-)50
STA 67+90
INV (-) 60.00
MATCH LINE, STA. 68+00
(-)40
64+00
65+00
66+00
67+00
68+00
PROFILE
SCALE: HOR = 1" = 100'
VER = 1" = 20'



PLAN
SCALE: 1"=100'



PROFILE
SCALE: HOR = 1"=100'
VER = 1"= 20'

TECHITE Technical Report

Title TECHITE® PIPE HYDRAULIC TESTS

Technical Report No.	01019 Rev. A
Date	December 17, 1971
Prepared By	M. Sayar
	B. Glascock

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Amoco Reinforced Plastics Company
3100 Jefferson Street, Riverside, California 92504

ENGINEERING REPORT
ER-01019
Revision A
December 17, 1971

TECHITE® PIPE HYDRAULIC TESTS

1.0 INTRODUCTION:

The information contained herein is a synopsis of the report written by Mr. Thomas Carmody, Associate Professor of Civil Engineering at the University of Arizona, relative to hydraulic tests performed on ten foot lengths of 12 inch diameter TECHITE® pipe. The tests were conducted at the University of Arizona during August, September and October, 1970. Copies of the original document are available upon request.

2.0 SUMMARY:

Results of the tests indicate the following:

2.1 Flow coefficients determined for pipe flowing full are:

Velocity (fps)	Manning "n"	Darcy Weisbach "f"	Hazen Williams "C"
4.0	.0092	.0160	146
5.0	.0092	.0155	145
6.0	.0091	.0154	144
7.0	.009	.0152	142.5
8.0	.009	.0149	142

2.2 Tests conducted on the pipe flowing partially full indicate a Manning's "n" of .010 to .009 for Reynolds number greater than 3×10^5. The effect of maximum misalignment is negligible (see Figure 2b).

2.3 The combined effects of wall roughness and normal joint losses cause the pipe to behave similarly to one with an equivalent sand roughness of 1/5000 (see Figure 2a).

2.0 SUMMARY: (Cont'd)

2.4 The hydraulic performance of a piping system is affected by workmanship during installation, but precise alignment of TECHITE pipe sections is not required to achieve good performance. A commercially unacceptable alignment (all joints misaligned with an average joining pull of 3.5°) was tested with only a 5 - 10% increase in losses. Poor installation of one joint (1 ¼ inch joint gap instead of the nominal 1/8 inch) yielded negligible head loss at an average flow velocity of 4.0 ft/sec.

3.0 CONCLUSIONS:

The hydraulic performance of the pipe of 12 inch diameter meets or exceeds the claims of the manufacturer ($n = 0.010$; $C_w = 145$). Since absolute wall roughness is constant for all pipe sizes (a function of the manufacturing process), it can be expected that losses from wall friction will be less in the larger diameter pipe. Joint losses can certainly be reduced by using 20 foot rather than 10 foot lengths.

4.0 TEST PROCEDURES AND RESULTS:

The piping tested consisted of nine 10 foot long sections of 12 inch diameter TECHITE pipe assembled to form a continuous 90 foot run. Water was dumped from a large sump to an overhead constant head tank. The water was supplied to the piping by way of a 12 inch galvanized approach line and 12 inch gate valve and a vertical "S" section. The water then flowed through two three foot wide open channels, the lower of which contained a 1.85 foot high sharp crested weir. The water then returned to the sump to complete the cycle. Flow rate was determined by measuring the weir head:

$$Q = bC_d \, 2/3 \sqrt{2g} \; h^{3/2}$$

where $b = 3'$, $C_d = 0.611 + .075 \frac{h}{w}$, $(w = 1.85')$

In the full pipe tests, piezometric head was measured at 18 points, two in each pipe. Data obtained from two of the 18 points (stations 16 and 76) was discarded due to faulty tapping of the pipe. The Darcy Weisbach equation $\Delta H = f \frac{L}{D} \frac{V^2}{2g}$ was

4.0 TEST PROCEDURES AND RESULTS: (Cont'd)

evaluated in the full pipe flow test. It was assumed that for a given pipe and fluid viscosity, the friction factor (f) does not change with velocity once turbulent flow is established. Each station head reading was subtracted from the reference point (H_6) to yield a ΔH or slope. By plotting this $\frac{\Delta H}{V^2/2g}$ vs. station and drawing, a "best fit" line to the points, "f" was determined directly (see Figure 1).

Hazen-Williams C and Manning n were computed from Darcy-Weisbach f as follows:

DARCY-WEISBACH: $$\Delta H = f \frac{L}{D}\frac{V^2}{2g}\text{, where } \frac{\Delta H}{L} = S\text{ , } D = 4R$$

so $$V = 2(\sqrt{2g/f})\ R^{0.5}S^{0.5}$$

so $$2\sqrt{2g/f} = \frac{V}{R^{.5}S^{.5}}$$

HAZEN-WILLIAMS: $$V = 1.318\ C\ R^{.63}S^{.54}$$

so $$V = 1.318\ C\ R^{.13}S^{.04}\ (R^{.5}S^{.5})$$

and $$V/(R^{.5}S^{.5}) = 1.318\ C\ R^{.13}S^{.04} = 2\sqrt{2g/f}$$

MANNING: $$V = \frac{1.49}{n} R^{2/3}S^{1/2}$$

so $$V = \frac{1.49}{n} R^{1/6}(R^{.5}S^{.5})$$

and $$V/(R^{.5}S^{.5}) = \frac{1.49}{n} R^{1/6} = 2\sqrt{2g/f}$$

Now, for 12 inch pipe flowing full with $2g = 64.4\ f/S^2$, we have

$R = D/4 = .25$ feet, $R^{.167} = 0.794$, $R^{.13} = 0.835$, $\sqrt{2g} = 8.02$

so that

HAZEN-WILLIAMS: $$C = \frac{14.5}{\sqrt{f}\ S^{.04}}$$

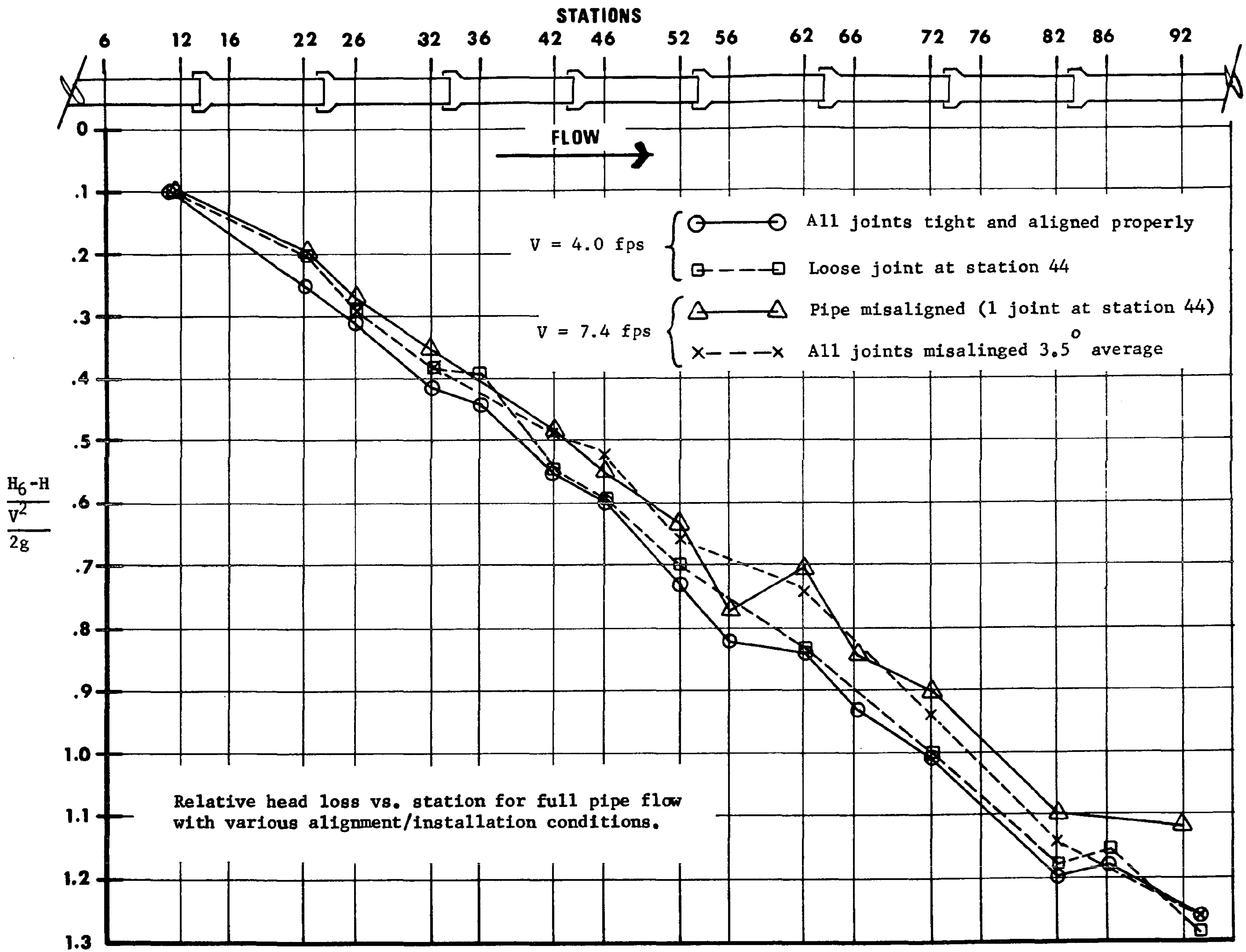
MANNING: $$n = 0.074\sqrt{f}$$

Plots of "f", "n" and "C" vs velocity are shown on Figure 2a.

4.0 TEST PROCEDURES AND RESULTS: (Cont'd)

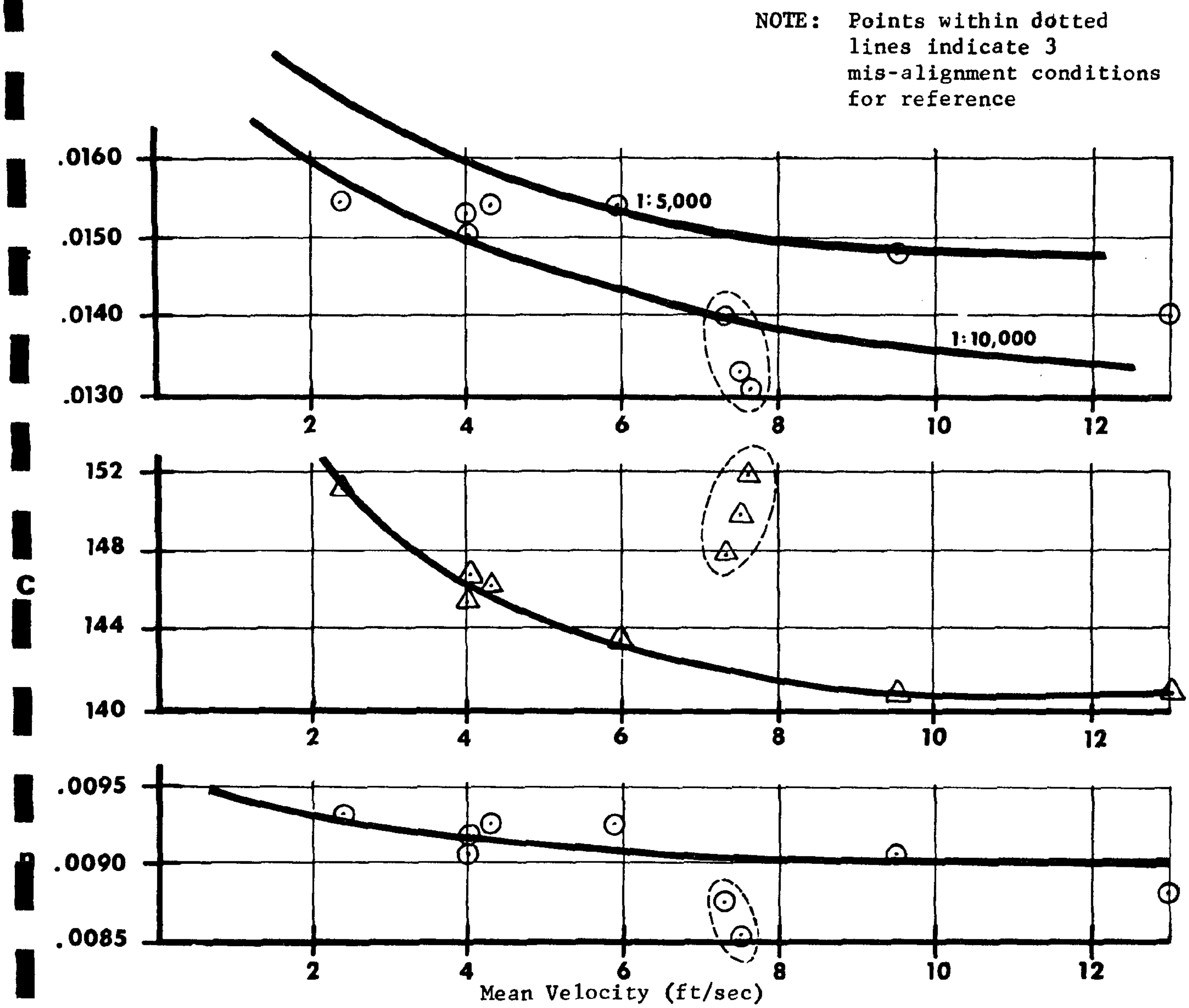
It is obvious that the Hazen Williams "C" should decrease as the velocity increases because the increase in slope of the piezometric head.

Partially full pipe flow tests were performed by adjusting the slope of the open channels to attain steady and uniform flow as evidenced by constant depth of flow. The value of Manning's "n" was then determined by referring to Table 103, Page 12 of "Steady Flow in Open Channels" by Woodwards and Posey (John Wiley and Sons, New York, Sixth Edition 1955).



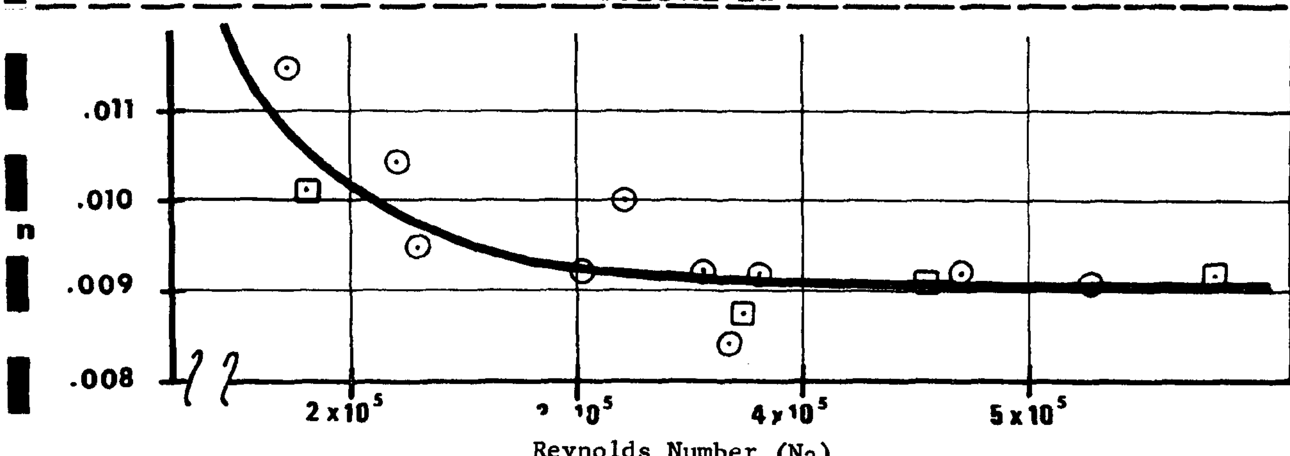
STATIONS
6
12
16
22
26
32
36
42
46
52
56
62
66
72
76
82
86
92
FLOW
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
$\frac{H_6 - H}{\frac{V^2}{2g}}$
V = 4.0 fps
All joints tight and aligned properly
Loose joint at station 44
V = 7.4 fps
Pipe misaligned (1 joint at station 44)
All joints misalinged 3.5° average
Relative head loss vs. station for full pipe flow with various alignment/installation conditions.

FIGURE I



Experimental values of f, C and vs. Mean Velocity - pipe flowing full

FIGURE 2a



Expiremental values of n vs. Reynolds Number - Partially filled pipe

FIGURE 2b

FIGURE 3

ALIGNMENT CONDITION FOR FLOW TEST

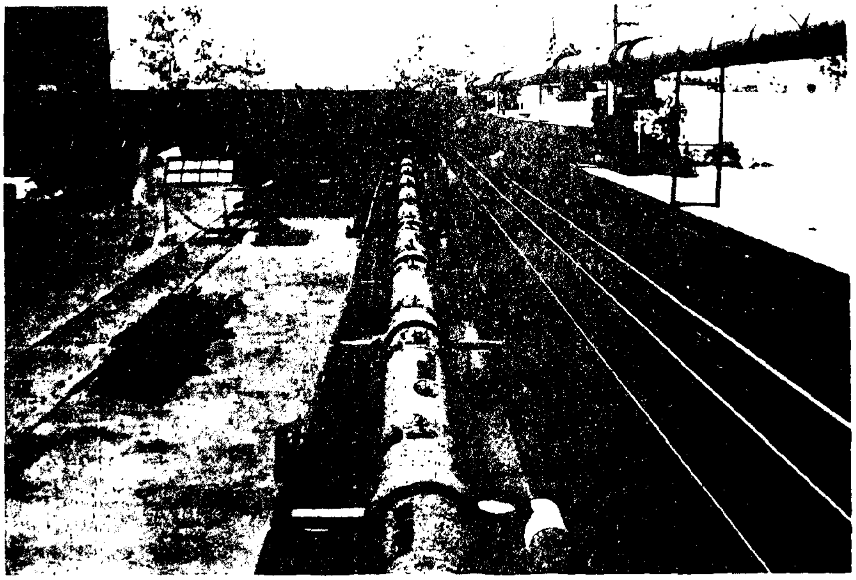


FIGURE 3-A STRAIGHT

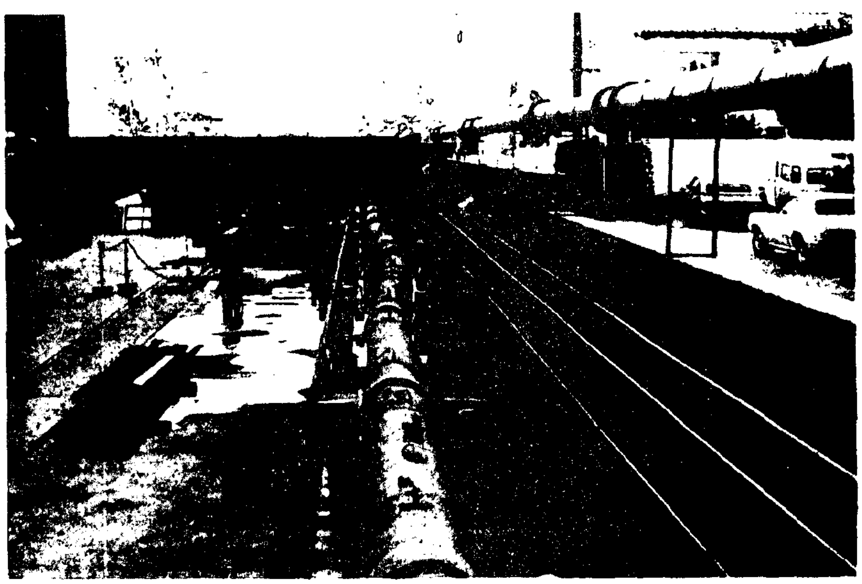


FIGURE 3-B WITH KINK AT MID-LENGTH

FIGURE 3 (Continued)

ALIGNMENT CONDITION FOR FLOW TEST

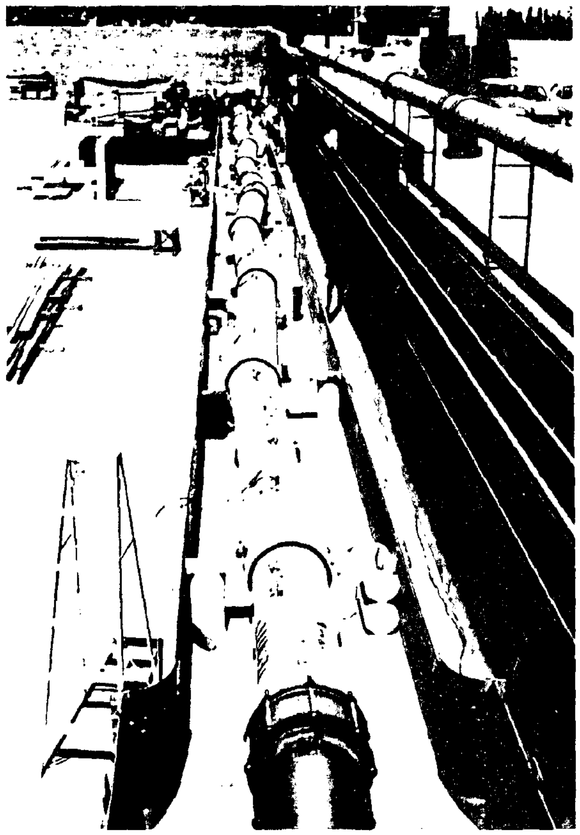


FIGURE 3-C EXTREMELY MISALIGNED

FIGURE 4

A TYPICAL PIEZOMETER TAP

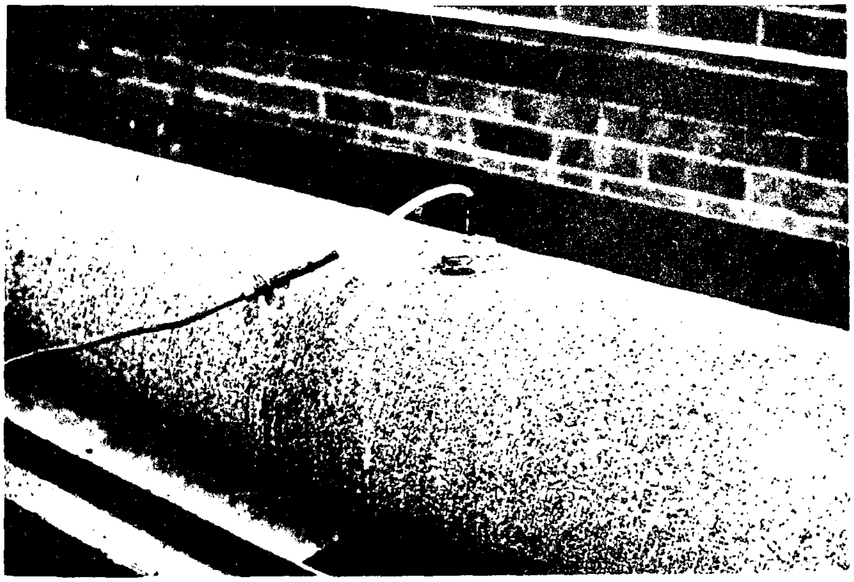
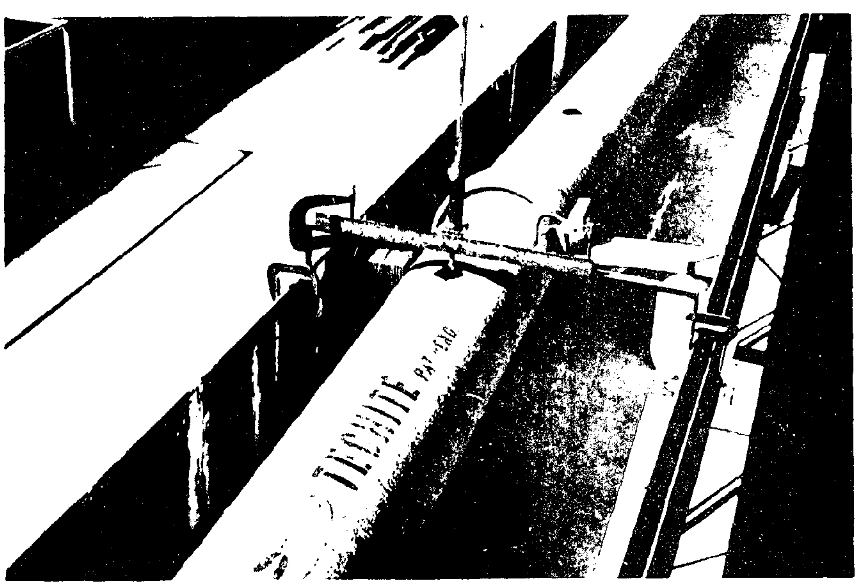


FIGURE 5

A TYPICAL POINT-GAUGE INSTALLATION



APPENDIX E

ACKNOWLEDGEMENTS

This report was prepared by Pacific Energy Management Consultants; however, other individuals, firms, and government agencies contributed with suggestions, reference material, engineering calculations and analyses, and technical performance data. This report would not be complete without the acknowledgement of their contribution:

I. PUMP MANUFACTURERS: 128 pump manufacturers were queried concerning the use of centrifugal pumps in a reverse mode as turbines. Among those responding, of particular help were the following:

Allis-Chalmers
Box 712
York, Pennsylvania 17405

Bell & Gossett ITT
8200 N. Austin Avenue
Morton Grove, Illinois 60053

Bingham-Willamette Company
2800 N.W. Front Avenue
Portland Oregon 97210

Buffalo Forge Company
P.O. Box 985
Buffalo, NY 14240

Crane Demings Pumps
884 South Boradway
Salem, Ohio 44460

Central Scientific Company, Inc.
2600 S. Kostner Ave.
Chicago, Illinois 60623

Challenge Manufacturing Co., Inc.
1308 67th Street
Oakland, California 94608

Dean Brothers Pumps, Inc.
P.O. Box 68172
Indianapolis, Indiana 46268

Dresser Industries, Inc.
5715 Bickett Street
Huntington Park, California 90255

Ecodyne, Smith & Loveless Division
14040 Santa Fe Trail Drive
Lenexa, Kansas 66215

Enpo Pump Co.
420 E. Third St.
Piqua, Ohio 45356

Galligher
P.O. Box 209
Salt Lake City, Utah 84110,

Gelber Pumps, Inc.
11090-G Artesia Blvd.
Cerritos, California 90701

Gorman-Rupp Company
P.O. Box 1217
Mansfield, Ohio 44903

ITT
4711 Golf Road
Skokie, Illinois 60076

LaBour Pump Co.
1607 Sterling Ave.
Elkhart, IN 46514

Morris Pumps, Inc.
Baldwinsville, New York 13027

Neptune Chemical Pump Co.
Lansdale, PA 19446

Paco-Pacific Pumping Co.
P.O. Box 12924
Oakland, CA 94604

Peabody Floway
P.O. Box 164
Fresno, California 93707

Piper Hydro Incorporated
3031 East Coronado
Anaheim, California 92806

Price Pump Co.
P.O. Box Q
Sonoma, CA 95476

Smith Precision
P.O. Box 276
Newbury Park, California 92320

Tuthill Corporation
12500 South Pulaski Road
Chicago, Illinois 60658

Weil Pump Company
1530 North Fremont Street
Chicago, Illinois 60622

Wilden Exports, Inc.
P.O. Box 845
Colton, California 92324

Wilfley
P.O. Box 2330
Denver, Colorado 80201

Wilson Snyder Pumps
P.O. Box 478
Dallas, Texas 75221

Worthington
14 Fourth Avenue
East Orange, NJ 07017

II. GUAM ENVIRONMENTAL PROTECTION AGENCY, Ricardo C. Duenas, Administrator: Of particular assistance in providing data and guidance for this report were:

James B. Branch, Deputy Administrator

John R. Worlund, Director, Water Division

Kenneth L. Morphew, Head, Monitoring Services Division

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Moscow, Idaho 83843

John P. Duenas, P.E.
Chief Engineer, Juan C. Tenorio and Associates
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Tamuning, Guam 96911

APPENDIX F

ECONOMIC ANALYSIS

An economic analysis has two principle goals:

* The estimation, in economic terms of the impact of a particular investment in energy conservation.

* The comparison and ranking of various alternatives available to an organization for future conservation.

The analysis is more accurate in the latter role rather than the former; estimation of future economic trends is at best educated guessing and thus the prediction of savings to be realized by a particular investment can only be approximate. However, various alternatives can validly be compared and ranked as long as estimates concerning future trends are held constant across all alternatives. It is important to bear this fact in mind when studying economic projections.

The basic principle of the analysis is that money has value (which changes) over time. An amount of money may be invested, banked or otherwise used, thus causing it to appreciate. $1,000 invested at 10% now will be worth $1,610.51 in 5 years (compounded annually). Similarly, money we expect to obtain at some future date is not worth its full amount in terms of today's dollars. $1,000 which we will obtain 6 years from now is only worth $564.47 today (assuming money grows 10% annually), because $564.47 invested today at 10% will, in 6 years, grow to $1,000.

Investments in energy conservation will realize savings at various times in the future; the only fair way to examine the economic impact of these investments is to adjust the amounts which we expect to obtain and express them all at a common time. Usually, future amounts are adjusted back to the present and expressed in terms of "present" dollars. This practice is called discounting.

It is easy to see that the result of an economic analysis is not the expression of the actual dollars saved over the lifetime of the investment; rather, it is a representation of the <u>value</u> of all savings in terms of today's money.

The income from this project takes the form of monthly dollar amounts which accrue over several years. Each increment is assumed to grow over the lifetime of the project at an annual rate. The cost of energy is assumed to change periodically. Thus, the monthly increment is not a constant over the lifetime of the investment, but will itself change at some annual rate. At the end of the project life a number of incremental monthly dollar amounts which have grown because of money's future worth and the changing value of the increment. But this amount accrued is at some future date. The accepted practice is to express this future amount in terms of present dollars. The analyses performed in this study incorporate these considerations to arrive at a present value of all income accrued.

Annual operating costs are estimated according to accepted engineering practice and discounted. The sum of all such costs are expressed in terms of present dollars. These costs will change annually, the result of a rate of inflation. This change is accounted for in the calculation. The present value of all O & M expenses over project life is found.

APPENDIX G

TYPICAL PUMP-GENERATOR

PRICE QUOTATIONS

I. IDAHO PUMP SUPPLY

Idaho Pump Supply is a firm experienced in reverse mode operation of pumps, having installed such devices on municipal wastewater disposal systems. Their basic quote involves supply of either a synchronous or induction generator. The system proposed is a short coupled verticle turbine consisting of:

* 8" x 16" discharge head assembly.
* 5' x 8" x 2 1/2" x 1 1/2" column tube and shaft.
* 3 stage 12B bowl assembly.
* 60 hp, U.H.S. gearhead with 1:1 ratio.
* Watson-Spicer drive shaft, model SL-36.
* 50 KW, 3 phase, 60 hertz AC generator.
* 3 minute electric motorized valve.
* Nema size #4 combination starter with circuit breaker disconnect, magnetic starter, voltage meter, amperage and frequency meter.
* Solid state frequency regulator.

Quote[1], sychronous generator: $15,324; induction generator: $13,950.

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[1]F.O.B. Twin Falls, Idaho.

II. BINGHAM-WILLAMETTE COMPANY

The Bingham-Willamette Company is an established manufacturer of pumps and turbines. Their quote involves an induction generator (but does not include valving):

* 4 x 6 x 7.5 HT-CAP Turbine.
* Westinghouse model 20-5H4-TBFC-KKN motor (generator).
* Export create.
* Spares.

Quote[2]: induction generator: $12,341.

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[2]F.O.B. Shreveport, Louisiana.

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